

CHAPTER 500 TRAFFIC INTERCHANGES

Topic 501 - General

Index 501.1 - Concepts

A traffic interchange is a combination of ramps and grade separations at the junction of two or more highways for the purpose of reducing or eliminating traffic conflicts, to improve safety, and increase traffic capacity. Crossing conflicts are eliminated by grade separations. Turning conflicts are either eliminated or minimized, depending upon the type of interchange design.

501.2 Warrants

All connections to freeways are by traffic interchanges. An interchange or separation may be warranted as part of an expressway (or in special cases at the junction of two non-access controlled highways), to improve safety or eliminate a bottleneck, or where topography does not lend itself to the construction of an intersection.

501.3 Spacing

The minimum interchange spacing shall be 1.5 km in urban areas, 3.0 km in rural areas, and 3.0 km between freeway-to-freeway interchanges and local street interchanges. To improve operations of closely spaced interchanges the use of auxiliary lanes, grade separated ramps, collector distributor roads, and/or ramp metering may be warranted.

See Design Information Bulletin No. 77 for additional information on interchange spacing, including the procedural and documentation requirements to be fulfilled prior to requesting an exception to the above standards.

Topic 502 - Interchange Types

502.1 General

The selection of an interchange type and its design are influenced by many factors including the following: the speed, volume, and composition of

traffic to be served, the number of intersecting legs, the standards and arrangement of the local street system including traffic control devices, topography, right of way controls, local planning, proximity of adjacent interchanges, community impact, and cost. Even though interchanges are, of necessity, designed to fit specific conditions and controls, it is desirable that the pattern of interchange ramps along a freeway follow some degree of consistency. It is frequently desirable to rearrange portions of the local street system in connection with freeway construction in order to effect the most desirable overall plan of traffic service and community development.

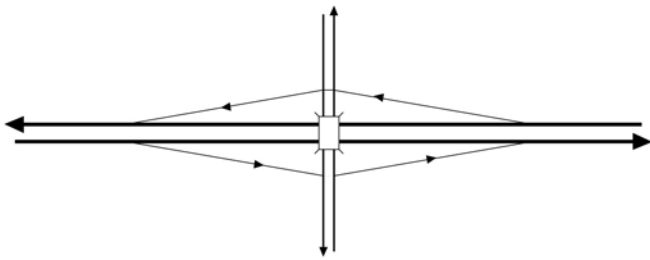
Interchange types are characterized by the basic shapes of ramps: namely, diamond, loop, directional, hook, or variations of these types. Many interchange designs are combinations of these basic types. Schematic interchange patterns are illustrated in Figure 502.2 and Figure 502.3. These are classified as: (a) Local street interchanges and (b) Freeway-to-freeway interchanges. See Chapter X of "A Policy on Geometric Design of Highways and Streets," AASHTO, for additional examples.

502.2 Local Street Interchanges

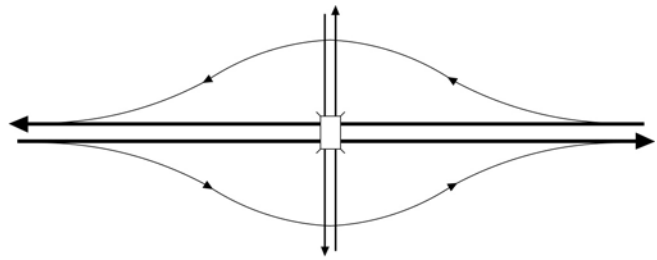
The use of isolated off ramps or partial interchanges should be avoided because of the potential for wrong-way movements and added driver confusion. In general, interchanges with all ramps connecting with a single cross street are preferred.

- (a) Diamond Interchange--The simplest form of interchange is the diamond. Diamond interchanges provide a high standard of ramp alignment, direct turning maneuvers at the crossroads, and usually have minimum construction costs. The diamond type is adaptable to a wide range of traffic volumes. The capacity is limited by the capacity of the intersection of the ramps at the crossroad. This capacity may be increased by widening the ramps to two or three lanes at the crossroad and by widening the crossroad in the intersection area. Crossroad widening will increase the length of under-crossings and the width of over-crossings, thus adding to the

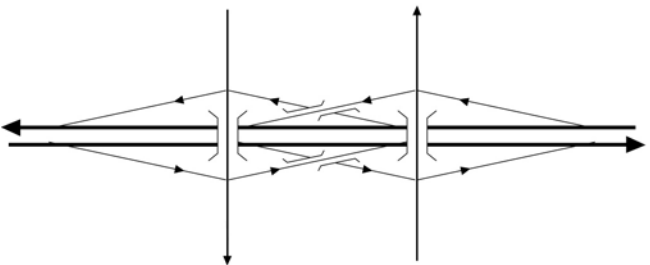
Figure 502.2
Typical Local Street Interchanges



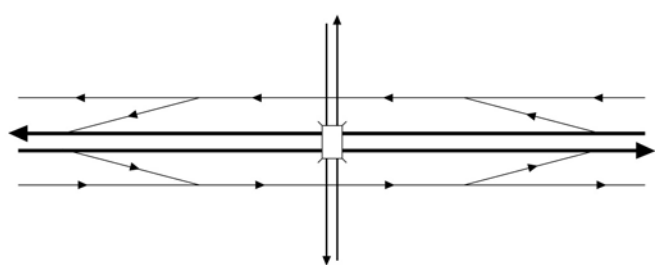
TYPE L-1



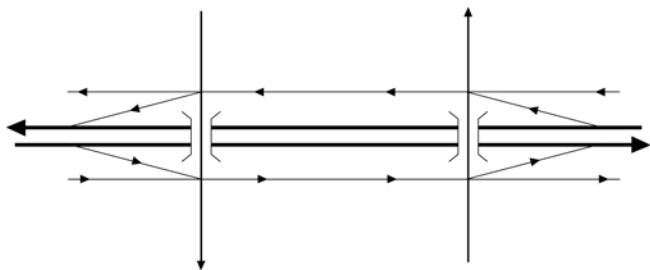
TYPE L-2



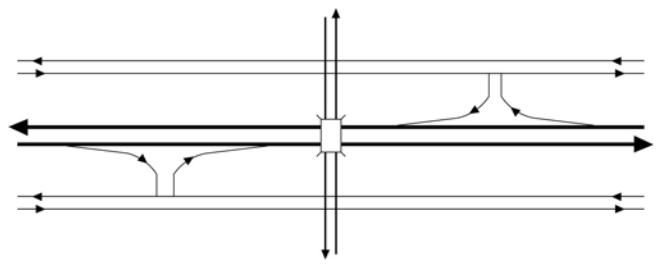
TYPE L-3



TYPE L-4

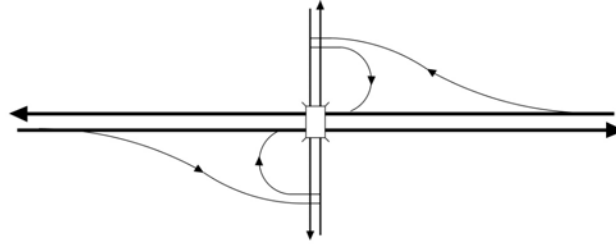


TYPE L-5

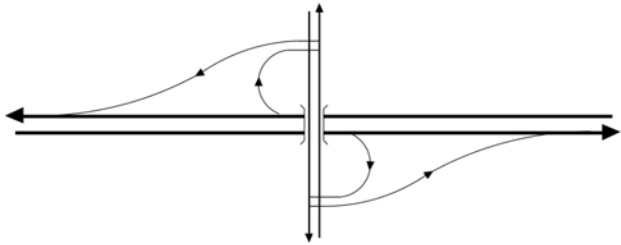


TYPE L-6

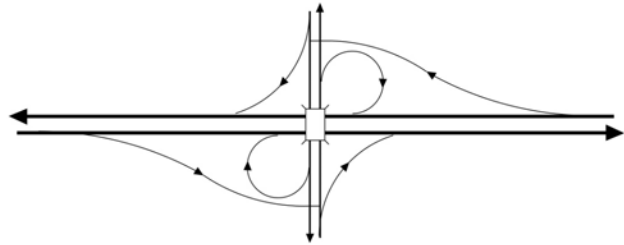
Figure 502.2
Typical Local Street Interchanges
(continued)



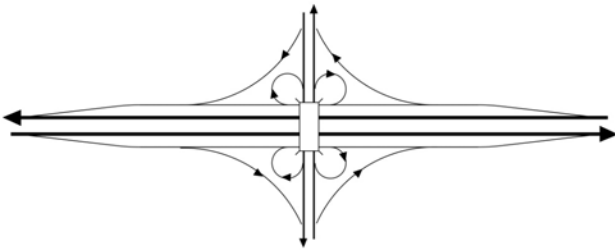
TYPE L-7



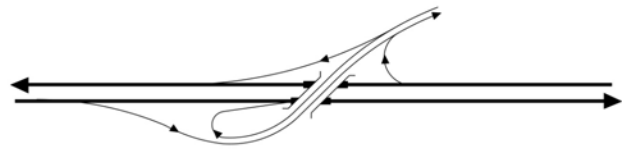
TYPE L-8



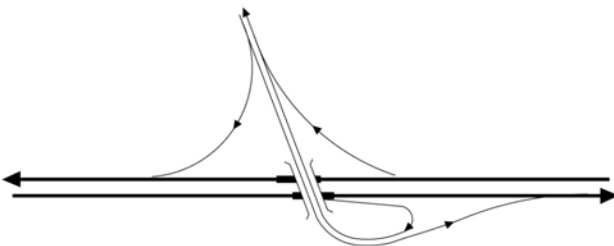
TYPE L-9



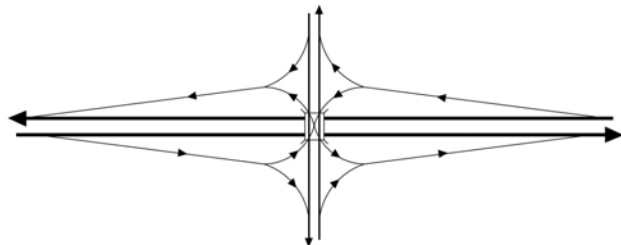
TYPE L-10



TYPE L-11



TYPE L-12



TYPE L-13

bridge cost. Ramp intersection capacity analysis is discussed in Topic 406.

The compact diamond (Type L-1) is most adaptable where the freeway is depressed or elevated and the cross street retains a straight profile. Type L-1's are suitable where physical, geometric or right of way restrictions do not permit a spread diamond configuration.

The spread diamond (Type L-2) is adaptable where the grade of the cross street is changed to pass over or under the freeway. The ramp terminals are spread in order to achieve maximum sight distance and minimum intersection cross slope, commensurate with construction and right of way costs, travel distance, and general appearance. A spread diamond has the advantage of flatter ramp grades, greater crossroads left-turn storage capacity, and the flexibility of permitting the construction of future loop ramps if required.

The split diamond with braids (Type L-3) may be appropriate where two major crossroads are closely spaced.

- (b) Interchanges with Parallel Street Systems--Types L-4, L-5 and L-6 are interchange systems used where the freeway alignment is placed between parallel streets. Types L-4 and L-5 are used where the parallel streets will operate with one-way traffic. In Type L-4 slip ramps merge with the frontage street and in Type L-5 the ramps terminate at the intersection of the frontage road with the cross street, forming five-legged intersections. In Type L-6 the freeway ramps connect with two-way parallel streets. The parallel streets in the Types L-4, L-5 and L-6 situation are usually too close to the freeway to permit ramp intersections on the cross street between the parallel frontage streets.

The "hook" ramps of the Type L-6 are often forced into tight situations that lead to less than desirable geometrics. The radius of the curve at the approach to the

intersection should exceed 50 m and a tangent of at least 50 m should be provided between the last curve on the ramp and the ramp terminal. Special attention should always be given to exit ramps that end in a hook to ensure that adequate sight distance around the curve, deceleration prior to the curve or end of anticipated queue, and adequate superelevation for anticipated driving speeds can be developed.

- (c) Cloverleaf Interchanges--The simplest cloverleaf interchange is the two-quadrant cloverleaf, Type L-7 or Type L-8, or a combination where the two loops are on the same side of the cross street. Type L-7 eliminates the need for left-turn storage lanes, on or under the structure, thus reducing the structure costs. These interchanges should be used only in connection with controls which preclude the use of diamond ramps in all four quadrants. These controls include right of way controls, a railroad track paralleling the cross street, and a short weaving distance to the next interchange.

The Type L-9, partial cloverleaf interchange, provides loop on-ramps in addition to the four diamond-type ramps. This interchange is suitable for large volume turning movements. Left-turn movements from the crossroads are eliminated, thereby permitting two-phase operation at the ramp intersections when signalized. Because of this feature, the Type L-9 interchange usually has capacity to handle the volume of interchange traffic which can be accommodated on the crossroads.

The four-quadrant cloverleaf interchange (Type L-10) has free-flow characteristics for all movements. It has the disadvantage of a higher cost than a diamond or partial cloverleaf design and a relatively short weaving section between the loop ramps which limits capacity. Collector-distributor roads should be incorporated in the design of four-quadrant cloverleaf

interchanges to separate the weaving conflicts from the through freeway traffic.

- (d) Trumpet Interchanges--A trumpet design, Type L-11 or L-12, may be used when a crossroads terminates at a freeway. This design should not be used if future extension of the crossroads is probable. The diamond interchange is preferable if future extension of the crossroads is expected.

- (e) Single Point Interchange (SPI)--The Type L-13 is a concept which essentially combines two separate diamond ramp intersections into one large at-grade intersection. It is also known as an urban interchange. Detailed information on SPI's is provided in the Single Point Interchange Planning, Design and Operational Guidelines (SPI Guidelines), originally issued by memorandum on June 15, 2001. Per the SPI Guidelines, the Project Development Coordinator and the Traffic Liaison must approve the SPI concept.

Type L-13 requires approximately the same right of way as the compact diamond. However, the construction cost is substantially higher due to the structure requirements. The capacity of the L-13 can exceed that of a compact diamond if long signal times can be provided and left turning volumes are balanced.

This additional capacity may be offset if nearby intersection queues interfere with weaving and storage between intersections. The disadvantages of the L-13 are: 1) Future expansion of the interchange is extremely difficult; 2) Stage construction for retrofit situations is costly; 3) Long structure spans require higher than normal profiles and deeper structure depths; and 4) Poor bicycle and pedestrian circulation.

502.3 Freeway-to-freeway Interchanges

- (1) *General.* The function of the freeway-to-freeway interchange is to link freeway

segments together so as to provide the optimum highway system. Parameters such as cost, environment, community values, traffic volumes, route continuity, map relatability, and safety should all be considered. Both the sign route and the major traffic volume should be to the left at a freeway-to-freeway interchange, if possible.

(2) *Design Considerations.*

- (a) *Cost*--The differential cost between interchange types is often significant. A cost-effective approach will tend to assure that an interchange is neither over nor underdesigned. Decisions as to the relative values of the previously mentioned parameters must be consistent with decisions reached on adjacent main line freeways.
- (b) *System Balance*--The freeway-to-freeway interchange is a critical link in the total freeway system. The level of traffic service provided will have impact upon the mobility and overall effectiveness of the entire roadway system. For instance, traffic patterns will adjust to avoid repetitive bottlenecks, and to the greatest degree possible, to temporary closures, accidents, etc. The freeway-to-freeway interchange should provide flexibility to respond to these needs so as to maximize the cost effectiveness of the total system.
- (c) *Elimination of Connections*--Freeway-to-freeway interchanges need not include all possible turning movements. Connections serving minor traffic volumes or significantly out-of-direction traffic movements should be omitted unless it can be demonstrated that traffic service and other benefits justify the costs. Considerations include:
 - *Traffic volumes*--Turning traffic volumes may be nominal or a small percentage of the total interchanging traffic.
 - *Circuitry*--Connections may only serve significantly out-of-direction traffic movements.

- Freeway location--Where three freeways cross so as to form a relatively small triangle, the omission of the backward freeway-to-freeway connections from one leg of the triangle to another may have little negative impact on local or through traffic service.
 - Use of local streets--Low turning volumes may be accommodated reasonably well by way of local interchanges and the local street system. There may be both traffic operational advantages and economic savings from utilizing and improving this local system in lieu of providing the freeway-to-freeway connections.
 - Staging--Staging possibilities should be thoroughly assessed. Provisions should be made for adding or upgrading ramps and connectors at a later time. For example, an initial loop ramp might be later upgraded to direct connector.
 - Effect on other traffic movements--Provision of minor movements may be detrimental to traffic operation on major branch connections and the main line freeways.
 - Costs--All construction and right of way savings and costs attributable to the elimination of turning movements should be considered. This includes possible additional local interchange and street costs as well as reductions in the freeway-to-freeway interchange costs.
 - Signing--Freeway-to-freeway traffic may be signed via the local street system. Routes should be sufficiently direct and well oriented to insure that the unfamiliar driver can follow them.
- (d) Local Traffic Service--In metropolitan areas a freeway-to-freeway interchange is usually superimposed over an existing street system. Local and through traffic requirements are often in conflict.

Combinations of local and freeway-to-freeway interchanges can result in designs that are both costly and so complex that the important design concepts of simplicity and consistency are compromised. Therefore, alternate plans separating local and freeway-to-freeway interchanges should be fully explored. Less than desirable local interchange spacing may result; however, this may be compensated for by upgrading the adjacent local interchanges and street system.

Local traffic service interchanges should not be located within freeway-to-freeway interchanges unless geometric standards and level of service will be substantially maintained.

- (e) Alignment--It is not considered practical to establish fixed freeway-to-freeway interchange alignment standards. An interchange must be designed to fit into its environment. Alignment is often controlled by external factors such as terrain, buildings, street patterns, route adoptions, and community value considerations. Normally, loops have radii in the range of 50 m to 65 m and direct connections should have minimum radii of 260 m. Larger radii may be proper in situations where the skew or other site conditions will result in minimal increased costs. Direct connection radii of at least 350 m are desirable from a traffic operational standpoint. High alignment and sight distance standards should be provided where possible.

Drivers have been conditioned to expect a certain standard of excellence on California freeways. The designer's challenge is to provide the highest possible standards consistent with cost and level of service.

- (3) *Types.* Several freeway-to-freeway interchange design configurations are shown on Figure 502.3. Many combinations and variations may be formed from these basic interchange types.

(a) Four-Level-Interchange--Direct

connections are appropriate in lieu of loops when required by traffic demands or other specific site conditions. The Type F-1 interchange with all direct connections provides the maximum in mobility and safety. However, the high costs associated with this design require that the benefits be fully substantiated.

The Type F-1 Alternative "A" interchange utilizes a single divergence ramp for traffic bound for the other freeway; then provides a secondary directional split. Each entrance ramp on a Type F-1A interchange is provided separately. The advantages of the Type F-1A are: 1) reduced driver confusion since there is only one exit to the other freeway, and 2) operations at the entrance may be improved since the ramps merge with the mainline one at a time.

The Type F-1 Alternative "B" interchange provides separate directional exit ramps and then merges the entering traffic into a single ramp before converging with the mainline. Since the Type F-1B combines traffic from two ramps before entering the freeway, it is important to verify that adequate weaving capacity is provided beyond the entrance. Separating the directional split of exiting traffic reduces the volume to each of the two ramps and therefore may improve the level of service of the weave section prior to the exit.

Design for a four-level interchange may combine the configuration of the Type F1-A and F1-B interchange to best suit the conditions at a given location.

(b) Combination Interchanges--The three-quadrant cloverleaf, Type F-2, with one direct connection may be necessary where a single move carries too much traffic for a loop ramp or where the one quadrant is restricted by environmental, topographic, or right of way controls.

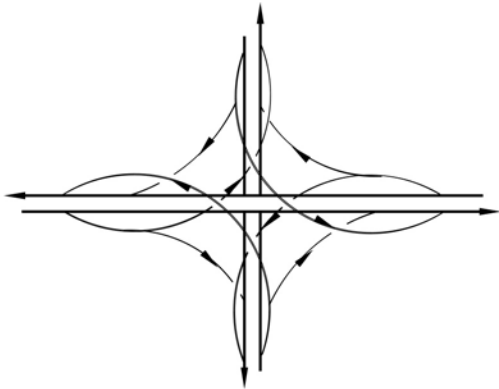
The two-loop, two-direct connection interchange, Type F-3, is often an appropriate solution. The weaving

conflicts which ordinarily constitute the most restrictive traffic constraint are eliminated, yet cost and right of way requirements may be kept within reasonable bounds. Consideration should be given to providing an auxiliary lane in advance of the loop off-ramps to provide for vehicle deceleration.

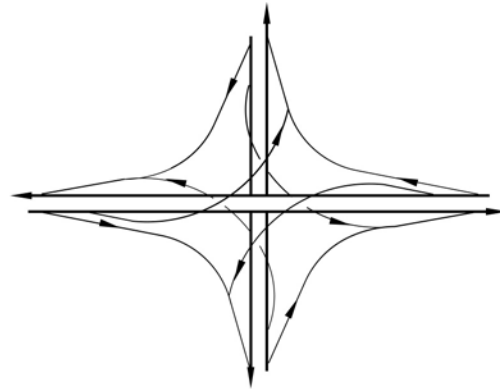
(c) Four-Quadrant Cloverleaf--The four-quadrant cloverleaf with collector-distributor roads, Type F-4, is ordinarily the most economical freeway-to-freeway interchange solution when all turning movements are provided. The four-quadrant cloverleaf is generally applicable in situations where turning volumes are low enough to be accommodated in the short weaving sections. It should be designed with collector-distributor roads to separate weaving conflicts from the through freeway traffic.

(d) Freeway Terminal Junction--Types F-5, F-6, F-7, and F-8 are examples of interchange designs where one freeway terminates at the junction with another freeway. In general, the standard of alignment provided on the left or median lane connection from the terminating freeway should equal or approach as near as possible that of the terminating freeway. Terminating the median lane on a loop should be avoided. It is preferable that both the sign route and the major traffic volume be to the left at the branch connection diverge. The choice between Types F-7 and F-8 should include considerations of traffic volumes, route continuity, and map relatability. When these considerations are in conflict, the choice is made on the basis of judgment of their relative merits.

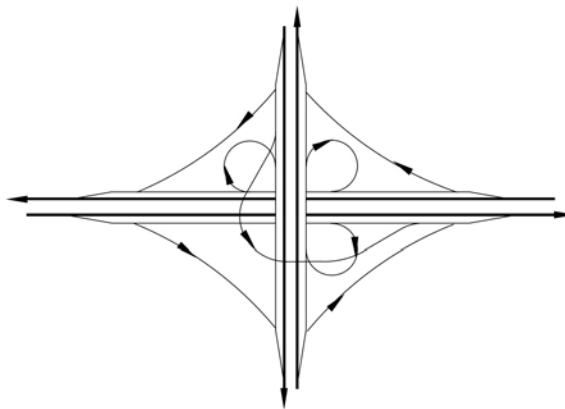
Figure 502.3
Typical Freeway-to-freeway
Interchanges



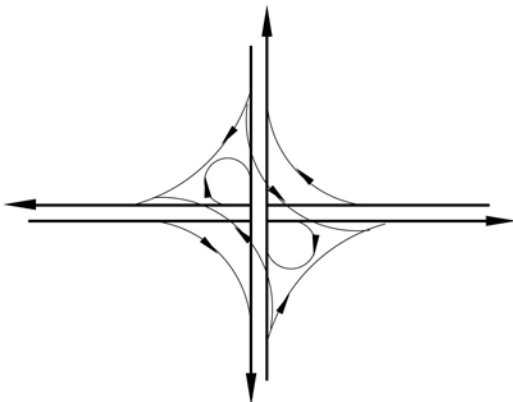
TYPE F-1 (ALT "A")



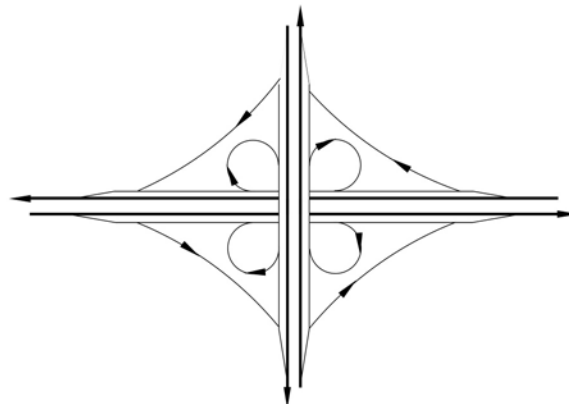
TYPE F-1 (ALT "B")



TYPE F-2

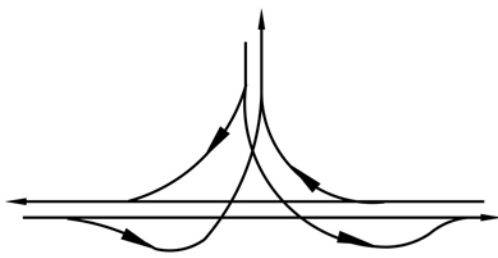


TYPE F-3

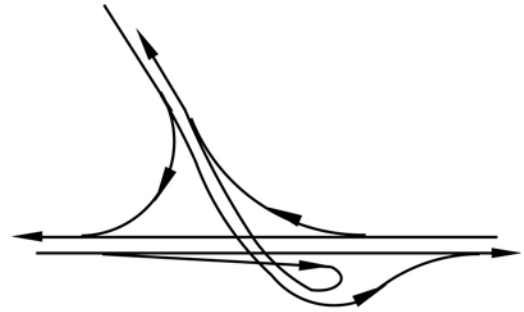


TYPE F-4

Figure 502.3
Typical Freeway-to-freeway Interchanges
(continued)



TYPE F-5



TYPE F-6



TYPE F-7



TYPE F-8

Topic 503 - Interchange Design Procedure

503.1 Basic Data

Data relative to community service, traffic, physical and economic factors, and potential area development which may materially affect design, should be obtained prior to interchange design. Specifically, the following information should be available:

- (a) The location and standards of existing and proposed local streets including types of traffic control.
- (b) Existing and proposed land use including such developments as shopping centers, recreational facilities, housing developments, schools, and other institutions.
- (c) A traffic flow diagram showing average daily traffic and design hourly volumes, as well as time of day (a.m. or p.m.), anticipated on the freeway ramps and affected local streets or roads.
- (d) The relationship with adjacent interchanges.
- (e) The location of major utilities, railroads, or airports.

503.2 Reviews

Interchanges are among the major design features which are to be reviewed by the Design Coordinator and/or Design Reviewer, HQ Traffic Liaison, other Headquarters staff, and the FHWA Field Operations Engineer, as appropriate. Major design features include the freeway alignment, geometric cross section, location of separation structures, closing of local roads, frontage road construction, and work on local roads. Particularly close involvement should occur during preparation of the Project Study Report and Project Report (see the Project Development Procedures Manual). Such reviews can be particularly valuable when exceptions from advisory or mandatory design standards are being considered and alternatives are being sought.

Topic 504 - Interchange Design Standards

504.1 General

Topic 504 discusses the standards that pertain to both local service interchanges (various ramp configurations) and freeway-to-freeway connections. The design standards, policies and practices covered in Indexes 504.2, and 504.5 through 504.8 are typically common to both ramp and connector interchange types. Indexes 504.3 and 504.4 separately discuss ramp standards and freeway-to-freeway connector standards, respectively.

504.2 Freeway Entrances and Exits

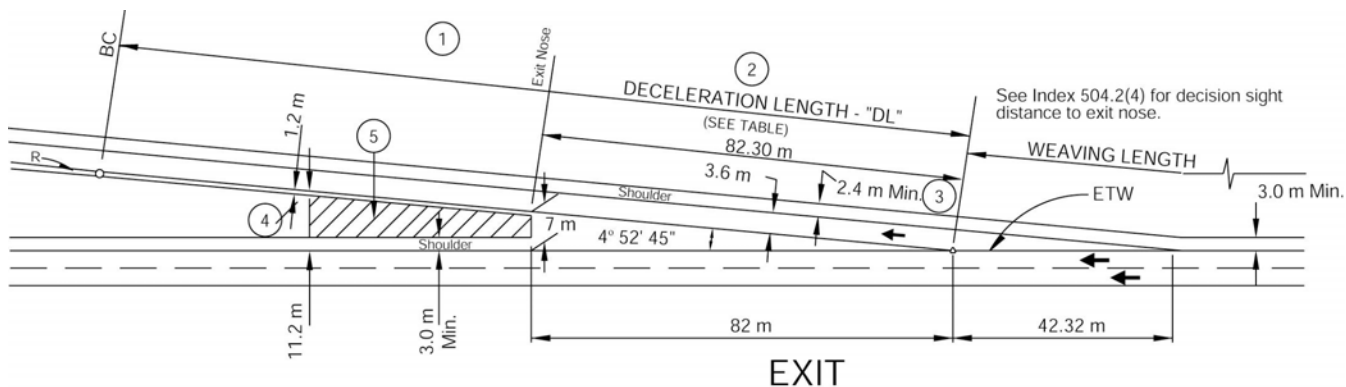
- (1) *Basic Policy.* **All freeway entrances and exits, except for direct connections with median High-Occupancy Vehicle lanes, shall connect to the right of through traffic.**
- (2) *Standard Designs.* Design of freeway entrances and exits should conform to the standard designs illustrated in Figure 504.2A-B (single lane), and Figure 504.3L (two-lane entrances and exits) and/or Figure 504.4 (diverging branch connections), as appropriate.

The minimum deceleration length shown on Figure 504.2B shall be provided prior to the first curve beyond the exit nose to assure adequate distance for vehicles to decelerate before entering the curve. The same standard should apply for the first curve after the exit from a collector-distributor road. The range of minimum "DL" (distance) vs. "R" (radius) is given in the table in Figure 504.2B. Strong consideration should be given to lengthening the "DL" distance given in the table when the subsequent curve is a descending loop or hook ramp, or if the upstream condition is a sustained downgrade (see AASHTO, A Policy on Geometric Design of Highways and Streets, for additional information).

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- ① On freeway to freeway connections the right paved shoulder shall be 3 m. - Table 302.1
- ② On single- and two-lane freeway to freeway connections the left paved shoulder shall be 1.5 m. - Table 302.1
- ③ When freeway is not on tangent alignment, select radius to approximate same degree of convergence (see Index 504.2(3)).
- ④ Locate as if it were to be center of a 0.3 m radius curb nose.
- ⑤ 1:15 Flare, 15 m long - Table 405.4.
- ⑥ 2% superelevation may be acceptable for the 1000 m radius curve on entrance ramps.
- ⑦ Contrasting surface treatment beyond the gore pavement (See Index 504.2(2)) (Advisory standard)

Figure 504.2B
Single Lane Freeway Exit



R (m)	Min. DL (m)	②
Less than 90	180	
90 - 149	150	
150 - 299	130	
300 & over	82.3	

NOTES:

- ① Minimum length between exit nose and end of ramp is 160 m for full stop at end of ramp.
- ② "DL" distance should be lengthened for descending, short radius curves, or if entered from a sustained downgrade.
- ③ On freeway to freeway connections the right paved shoulder shall be 3 m. - Table 302.1
- ④ On single- and two-lane freeway to freeway connections the left paved shoulder shall be 1.5 m. - Table 302.1
- ⑤ Contrasting surface treatment beyond the gore pavement (See Index 504.2(2)) (Advisory standard)

The exit nose shown on Figure 504.2B may be located downstream of the 7 m dimension; however, the maximum paved width between the mainline and ramp shoulder edges should be 7 m. Also, see pavement cross slope requirements in Index 504.2(5).

Contrasting surface treatment beyond the gore pavement should be provided on both entrance and exit ramps as shown on Figures 504.2A, 504.2B, and 504.3L. This treatment can both enhance aesthetics and minimize maintenance efforts. It should be designed so that a driver will be able to identify and differentiate the contrasting surface treatment from the pavement areas that are intended for regular or occasional vehicular use (e.g., traveled way, shoulders, paved gore, etc.). Consult with the District Landscape Architect, District Materials Engineer, and District Maintenance Engineer to determine the appropriate contrasting surface treatment of the facility at a specific location.

Refer to the HOV Guidelines for additional information specific to direct connections to HOV lanes.

- (3) *Location on a Curve.* Freeway entrances and exits should be located on tangent sections wherever possible in order to provide maximum sight distance and optimum traffic operation. Where curve locations are necessary, the ramp entrance and exit tapers should be curved also. The radius of the exit taper should be about the same as the freeway edge of traveled way in order to develop the same degree of divergence as the standard design (see Figure 504.2C).

On entrance ramps the distance from the inlet nose (4.25-meter point) to the end of the acceleration lane taper should equal the sum of the distances shown on Figure 504.2A. The 50:1 taper may be curved to fit the conditions, and the 1000 m radius curve may be adjusted (see Figure 504.2A, note 5).

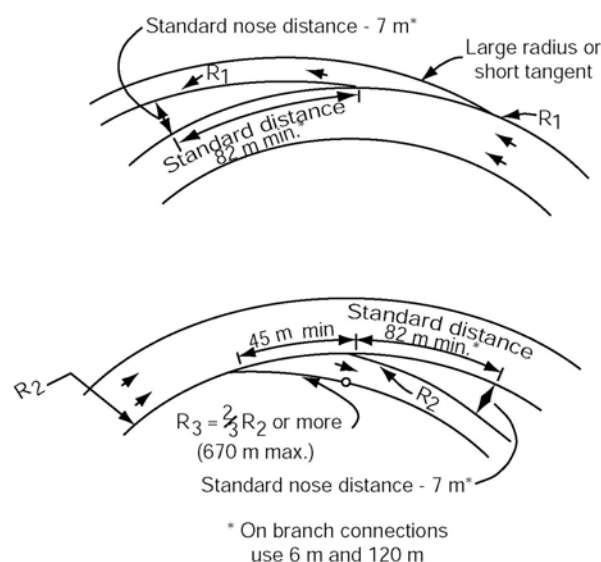
When an exit must be located where physical restrictions to visibility cannot be corrected by cut widening or object removal, an auxiliary lane in advance of the exit should be provided.

The length of auxiliary lane should be a minimum 180 m, 300 m preferred.

- (4) *Design Speed Considerations.* In the design of interchanges it is important to provide vertical and horizontal alignment standards which are consistent with driving conditions expected on branch connections. Sight distance on crest vertical curves should be consistent with expected approach speeds.

- (a) Freeway Exit--The design speed at the exit nose should be 80 km/h or greater for both ramps and branch connections.

**Figure 504.2C
Location of Freeway Ramps
on a Curve**



Decision sight distance given in Table 201.7 should be provided at freeway exits and branch connectors. At secondary exits on collector-distributor roads, a minimum of 190 m of decision sight distance should be provided. In all cases, sight distance is measured to the center of ramp lane right of the nose.

- (b) Freeway Entrance--The design speed at the inlet nose should be consistent with approach alignment standards. If the approach is a branch connection or diamond ramp with high alignment

standards, the design speed should be at least 80 km/h.

- (c) Ramps--See Index 504.3(1)(a).
- (d) Freeway-to-Freeway Connections -- See Index 504.4(2).

- (5) *Grades.* Grades for freeway entrances and exits are controlled primarily by the requirements of sight distance. Ramp profile grades should not exceed 8% with the exception of descending entrance ramps and ascending exit ramps, where a 1% steeper grade is allowed. However, the 1% steeper grade should be avoided on descending loops to minimize overdriving of the ramp (see Index 504.3 (8)).

Profile grade considerations are of particular concern through entrance and exit gore areas. In some instances the profile of the ramp or connector, or a combination of profile and cross slope, is sufficiently different than that of the freeway through lanes that grade breaks across the gore may become necessary. Where adjacent lanes or lanes and paved gore areas at freeway entrances and exits are not in the same plane, the algebraic difference in pavement cross slope should not exceed 5% (see Index 301.2). The paved gore area is typically that area between the diverging or converging edge of traveled ways and the 7 meter point.

In addition to the effects of terrain, grade lines are also controlled by structure clearances (see Indexes 204.6 and 309.2). Grade lines for overcrossing and undercrossing roadways should conform to the requirements of HDM Topic 104 Roads Under Other Jurisdictions.

- (a) Freeway Exits--Vertical curves located just beyond the exit nose should be designed with a minimum 80 km/h stopping sight distance. Beyond this point, progressively lower design speeds may be used to accommodate loop ramps and other geometric features.

Ascending off-ramps should join the crossroads on a reasonably flat grade to expedite truck starts from a stopped condition. If the ramp ends in a crest vertical curve, the last 15 m of the ramp

should be on a 5% grade or less. There may be cases where a drainage feature is necessary to prevent crossroads water from draining onto the ramp.

On descending off-ramps, the sag vertical curve at the ramp terminal should be a minimum of 30 m in length.

- (b) Freeway Entrances--Entrance profiles should approximately parallel the profile of the freeway for at least 30 m prior to the inlet nose to provide intervisibility in merging situations. The vertical curve at the inlet nose should be consistent with approach alignment standards.

Where truck volumes (three-axle or more) exceed 20 per hour on ascending entrance ramps to freeways and expressways with sustained upgrades exceeding 2%, a 450 m length of auxiliary lane should be provided in order to insure satisfactory operating conditions. Additional length may be warranted based on the thorough analysis of the site specific grades, traffic volumes, and calculated speeds; and after consultation with representatives of the Headquarters Division of Traffic Operations and the Division of Design. Also, see Index 204.5 "Sustained Grades".

504.3 Ramps

(1) General.

- (a) Design Speed -- When ramps terminate at an intersection at which all traffic is expected to make a turning movement, the minimum design speed along the ramp should be 40 km/h. When a "through" movement is provided at the ramp terminus, the minimum ramp design speed should meet or exceed the design speed of the highway facility for which the through movement is provided. The design speed along the ramp will vary depending on alignment and controls at each end of the ramp. An acceptable approach is to set design speeds of 40 km/h and 80 km/h at the ramp terminus and exit nose,

respectively, the appropriate design speed for any intermediate point on the ramp is then based on its location relative to those two points. When short radius curves with relatively lower design speeds are used, the vertical sight distance should be consistent with approach vehicle speeds. See Index 504.2(4) for additional information regarding design speed for ramps.

- (b) **Lane Width--Ramp lanes shall be a minimum of 3.6 m in width. Where ramps have curve radii of 90 m or less, measured along the outside ETW for single lane ramps or along the outside lane line for multilane ramps, with a central angle greater than 60 degrees, the single ramp lane, or the lane furthest to the right if the ramp is multilane, shall be widened in accordance with Table 504.3A in order to accommodate large truck wheel paths** (see Topic 404). Consideration may be given to widening more than one lane on a multilane ramp with short radius curves if there is a likelihood of considerable bus or truck usage of that lane.

**Table 504.3A
Ramp Widening for Trucks**

Ramp Radius (m)	Widening (m)	Lane Width (m)
<40	2.0	5.6
40 - 44	1.6	5.2
45 - 54	1.3	4.9
55 - 64	0.9	4.5
65 - 74	0.6	4.2
75 - 90	0.3	3.9
>90	0	3.6

- (c) **Shoulder Width--Shoulder widths for ramps shall be as indicated in Table 302.1.** Typical ramp shoulder widths are 1.2 m on the left and 2.4 m on the right.
- (d) **Lane Drops--Typically, lane drops are to be accomplished over a distance equal to**

2/3WV. Where ramps are metered, the recommended lane drop taper past the meter limit line is 50 to 1. Where conditions preclude the use of a 50 to 1 taper, the lane should be dropped using a taper of no less than 30 to 1. **However, the lane drop taper past the limit line shall not be less than 15 to 1.**

Lane drop tapers should not extend beyond the 2-meter point (the beginning of the weaving length) without the provision of an auxiliary lane.

- (e) **Lane Additions --** Lane additions to ramps are usually accomplished by use of a 36 m bay taper. See Table 405.2A for the geometrics of bay tapers.
- (2) *Ramp Metering*

All geometric designs for ramp metering installations must be discussed with the Design Coordinator or Design Reviewer from the Division of Design. **Design features or elements which deviate from the mandatory standards require the approvals described in Index 82.2.** Before beginning any ramp meter design, the designer must contact District Traffic Operations for direction in the application of procedural requirements of the Division of Traffic Operations.

Geometric ramp design for new facilities should normally be based upon the projected peak-hour traffic volumes 20 years after completion of construction, except as stated in Index 103.2.

Geometric ramp design for operational improvement projects for ramp meters should be based on current peak-hour traffic volume (this is considered to be data that is less than two years old). If this data is not available it should be obtained before proceeding with design. Peak hour traffic data from the annual Traffic Volumes book is not adequate for this application.

The design advice and typical designs that follow should not be directly applied to ramp meter installation projects, especially retrofit

designs, without giving consideration to "customizing" the geometric design features to meet site and traffic conditions (i.e., design highway volume, geometry, speeds, etc.). Every effort should be made by the designer to exceed the recommended minimum standards provided herein, where conditions are not restrictive.

(a) Metered Single-Lane Entrance Ramps

Geometrics for a single-lane ramp meter should be provided for volumes up to 900 vehicles per hour (vph) (see Figures 504.3A and 504.3B). Where truck volumes (3-axle or more) are 5% or greater on ascending entrance ramps to freeways with sustained upgrades exceeding 3% (i.e., at least throughout the merge area), a minimum 150 m length of auxiliary lane should be provided beyond the ramp convergence point. For additional guidance see Table X-5 of "A Policy on Geometric Design of Highways and Streets", AASHTO.

A multi-lane ramp segment may be provided to increase vehicle storage within the available ramp length (see 504.3(2)(d) Storage Length) and/or to create a preferential lane for HOVs, as required in Section 504.3(2)(h).

(b) Metered Multi-Lane Entrance Ramps

When entrance ramp volumes exceed 900 vph, and/or when an HOV lane is determined to be necessary, a two or three lane ramp segment should be provided. Figures 504.3C, 504.3D and 504.3E illustrate typical designs for metered two-lane ramps; and Figures 504.3F and 504.3G illustrate typical designs for metered three lane ramps. On two-lane loop ramps, normally only the right lane needs to be widened to accommodate design vehicle off-tracking. See 504.3(1)(b).

Three-lane metered ramps are typically needed to serve peak (i.e., commute) hour traffic along urban and suburban freeway corridors. The adverse effects of bus and

truck traffic on the operation of these ramps (i.e., off-tracking, sight restriction, acceleration characteristics on upgrades, etc.) is minimized when the ramp alignment is tangential or consists of curve radii not less than 90 m.

The recommended widths for metered ramps are shown in Table 504.3B.

On local street entrance ramps, the multi-lane segment should transition to a single lane width between the ramp meter limit line and the 2 m separation point (from the mainline edge of traveled way). See Figures 504.3C, 504.3D, 504.3E, 504.3F, 504.3G, 504.3H and 504.3I.

**Table 504.3B
Pavement Widths**

Metered Ramp	Traveled Way	Inside Shoulder	Outside Shoulder
1-lane	3.6 m	1.2 m	2.4 m
2-lane	7.2 m	1.2 m	2.4 m
3-lane	10.8 m	0.6 m	0.6 m

The lane drop transition should be accomplished with a taper of 50:1 unless a lesser taper is warranted by site and/or project specific conditions which control the ramp geometry and/or anticipated maximum speed of ramp traffic. For example, "loop" entrance ramps would normally not allow traffic to attain speeds which would warrant a 50:1 lane drop taper. Also, in retrofit situations, existing physical, environmental or right of way constraints may make it impractical to provide a 50:1 taper, especially if the maximum anticipated approach speed will be less than 80 km/h. Therefore, depending on approach geometrics and speed, the lane drop transition should be accomplished with a taper of between 30 and 50:1. **However, the lane drop taper past the limit line shall not be less than 15 to 1.**

Where truck volumes (3-axle or more) are 5% or greater on ascending entrance ramps to freeways with sustained upgrades exceeding 3% (i.e. at least throughout the merge area), a minimum 300 m length of auxiliary lane should be provided beyond the ramp convergence point. Table X-5, "A Policy on Geometric Design of Highways and Streets", AASHTO, provides additional guidance on acceleration lane length on grades.

When ramp volumes exceed 1,500 vph, a 300 m minimum length of auxiliary lane should be provided beyond the ramp convergence point. If an auxiliary lane is included, the ramp lane transition may be extended to the convergence point. However, the proximity of the nearest interchange may warrant weaving analysis to determine the acceptability of extending the ramp lane transition beyond the 2 m separation point. A longer auxiliary lane should be considered where mainline/ramp gradients and truck volumes warrant additional length.

(c) Metered Freeway-to-Freeway Connectors

Freeway-to-freeway connectors may also be metered when warranted. The need to meter a freeway-to-freeway connector should be determined on an individual basis. Because connector ramps provide a link between two high speed facilities, drivers do not expect to stop, nor do they expect to approach a stopped vehicle.

The installation of ramp meters on connector ramps shall be limited to those facilities which meet or exceed the following geometric design criteria:

- **standard lane and shoulder widths**
- **"tail light" sight distance, measured from 1070 mm eye height to a 600 mm object height, is provided for a design speed of 80 km/h minimum**

All lane drop transitions on connectors shall be accomplished with a taper of 50:1 minimum (see Figures 504.3H and 504.3I).

(d) Storage Length

In keeping the Strategic Plan to maximize the effectiveness of operational strategies, an important design consideration for a ramp meter system is providing adequate storage for queues. The District Operations Branch responsible for ramp metering shall be consulted to determine the desirable ramp meter storage.

Ramp meters have practical lower and upper output limits of 240 and 900 vph per lane, respectively. Ramp meter signals set for flow rates outside this range tend to have high violation rates and cannot effectively control traffic. Therefore, on a ramp with peak hour volume between 500 and 900, a two-lane ramp meter may be provided to double the vehicles stored within the available storage area. A single-lane ramp meter should be used when rates are below 500 vph and no HOV preferential lane is provided.

To minimize the impact on local street operation, every effort should be made to meet the recommended storage length. Wherever feasible, ramp metering storage should be contained on the ramp by either widening or lengthening it. Improvements to the local street system in the vicinity of the ramp should also be thoroughly investigated where there is insufficient storage length on the ramp and the ramp queue will adversely affect local street operation. The storage length that can be provided on the ramp may be limited by the weaving distance to the next off-ramp and available right of way. Local street improvements can include widening or restriping the street(s) or intersection(s) to provide additional storage or capacity. Signal timing revisions along the corridor

feeding the ramp can also enhance the storage capability. These will require coordination with the local agency consistent with the regional traffic operations strategy. Ultimately system-wide adaptive ramp metering will coordinate with local street and arterial signal systems.

The current peak period 5, 6, or 15 minute arrival rates and anticipated or current ramp meter discharge rates should be used to determine the storage length required for ramp metering. It is recommended that a minimum vehicle spacing of 9 m be used for designing storage on metered ramps. Additional spacing should be provided for locations where there are significant percentages of trucks, buses, or recreational vehicles.

It is the responsibility of Caltrans, on Caltrans initiated projects, to mitigate the effect of ramp metering, for initial as well as future operational impacts, on local streets that intersect and feed entrance ramps to the freeway. Developers and/or local agencies, however, should be required to mitigate any impact to existing ramp meter facilities, future ramp meter installations, or local streets, when those impacts are attributable to new development and/or local agency roadway improvement projects.

(e) Structural Section

In planning for the possibility of future widening, the structural section for the ramp shoulders should be equal to the ramp traveled way structural section. In locations where failure of loop detectors due to asphalt concrete pavement deterioration is a concern, a Portland Cement Concrete (PCC) pad may be considered on new construction and rehabilitation projects. The concrete pad should cover the metering detector loop area upstream and downstream of the limit line.

(f) Meter Location

On single-lane ramps, the ramp meter signal standard should be placed on the driver's left.

(g) Limit Line Location

The limit line location will be determined by the selected transition taper, but should be a minimum of 23 m upstream of the 7 m point on the entrance ramp as shown in Figures 504.3A-I. A single 300 mm solid white line shall be placed across all metered lanes. Staggered limit lines shall not be used.

(h) HOV Preferential Lane

Ramp meter installations should operate in conjunction with, and complement other transportation management system elements and transportation modes. As such, ramp meter installations should include preferential treatment of carpools and transit riders. Specific treatment(s) must be tailored to the unique conditions at each ramp location, however the standard or base treatment upon which other strategies are designed is the High Occupancy Vehicle (HOV) preferential lane.

Division of Traffic Operations policy requires an HOV preferential lane be provided at all ramp meter locations. Deviation from this policy requires concurrence from the Traffic Liaison, which must be reflected in the Project Initiation Document.

In general, the vehicle occupancy requirement for ramp meter HOV preferential lanes will be two or more persons per vehicle. At some locations, a higher vehicle occupancy requirement may be necessary. The occupancy should be based on the HOV demand and coordination with other HOV facilities in the vicinity.

A preferential lane should typically be placed on the left, however demand and operational characteristics at the ramp entrance may dictate otherwise. The District Operations Branch responsible for ramp metering shall determine which side of the ramp they shall be placed, and whether or not the HOV lane will be metered.

- It is the policy of Districts 4, 6, 8, and 11 to meter the HOV preferential lane
- Districts 3, 7, and 12 typically do not meter the HOV preferential lane

Access to the HOV preferential lane may be provided in a variety of ways depending on interchange type and the adequacy of storage provided for queued vehicles. Where queued vehicles are expected to block access to the HOV preferential lane, direct or separate access should be considered. Designs should consider pedestrian/ bicycle volumes, especially when the entrance ramp is located near a school or the local highway facility includes a designated bicycle lane or route. Contact the Traffic Liaison and the Design Reviewer to discuss the application of specific design and/or general issues related to the design of HOV preferential lane access.

Signing for an HOV preferential lane should be placed to clearly indicate which lane is designated for HOVs. Real-time signing at the ramp entrance, such as an overhead extinguishable message sign, may be necessary at some locations if pavement delineation and normal signing do not provide drivers with adequate lane usage information. To avoid trapping Single Occupancy Vehicles (SOVs) in an HOV preferential lane, pavement delineation at the ramp entrance should lead drivers into the SOV lane.

(i) Modifications to Existing HOV Preferential Lanes

Changes in traffic conditions, proposals for interchange modifications, recurrent operational problems affecting the local facility, or the need to further improve mainline operations through more restrictive metering all provide an opportunity to reevaluate the need for an HOV preferential lane. HOV preferential lanes should remain in place or be added to the scope of projects generated in response to any of the above scenarios. Alternate solutions should be investigated before removal is considered. For example: Better control over ramp traffic can be attained by retrofitting ramps to meter HOV traffic which bypasses the ramp meter (District 3, 7, and 12). Underutilization of an existing lane plus the need for additional right of way for storage, the availability of an alternate HOV entrance ramp within 2 km, or the availability of a direct HOV access (drop) ramp will typically provide adequate justification for the removal of a preferential lane at specific locations.

The Deputy District Director of Operations, in consultation with the HQ Traffic Liaison, is responsible for approving decisions to remove HOV preferential lanes. Written documentation should be provided in the appropriate project document(s).

(j) Enforcement Areas and Maintenance Pullouts

Division of Traffic Operations policy requires an enforcement area be provided on all two-lane and three-lane on-ramps with HOV lanes. Deviation from this policy requires concurrence from the Traffic Liaison, which must be reflected in the Project Initiation Document.

On single-lane ramps, a paved enforcement area is not necessary but the area should be graded to facilitate future ramp widening (see Figure 504.3A). Enforcement areas are used by the California Highway Patrol

(CHP) to enforce vehicle occupancy requirements. At locations where the HOV lane is metered, the enforcement area should begin as close to the limit line as practical. Where unmetered, it should begin approximately 50 m downstream of the limit line. On three-lane ramps, the enforcement area should be downstream of the mast arm standard, approximately 21 m from the limit line. The length of the enforcement area and its distance downstream of the limit line may be adjusted to fit conditions at the ramp with CHP approval.

The District Traffic Operations Branch responsible for ramp metering shall coordinate enforcement issues with the California Highway Patrol. The CHP Area Commander shall be contacted during the Project Report stage, prior to design, to discuss any variations needed to the enforcement area designs shown in this manual. Variations shall be discussed with the Traffic Liaison and the Design Coordinator and/or Design Reviewer.

A paved pullout area near the controller cabinet (see Standard Plan H8) should be provided for safe and convenient access for Maintenance and Operations personnel. If a pullout cannot be provided, a paved or "all weather" walkway should be provided to the controller cabinet, see Index 107.2. See Topic 309, Clearances, for placement guidance of fixed objects such as controller cabinets.

(3) *Location and Design of Ramp Intersections on the Crossroads.*

Factors which influence the location of ramp intersections on the crossroads include sight distance, construction and right of way costs, circuitry of travel for left-turn movements, crossroads gradient at ramp intersections, storage requirements for left-turn movements off the crossroads, and the proximity of other local road intersections.

Ramp terminals should connect where the grade of the overcrossing is 4% or less to avoid potential overturning of trucks.

For left-turn maneuvers from an off-ramp at an unsignalized intersection, the length of crossroads open to view should be greater than the product of the prevailing speed of vehicles on the crossroads, and the time required for a stopped vehicle on the ramp to execute a left-turn maneuver. This time is estimated to be 7-1/2 seconds.

Where a separate right-turn lane is provided at ramp terminals, the turn lane should not continue as a "free" right unless pedestrian volumes are low, the right-turn lane continues as a separate full width lane for at least 60 m prior to merging and access control is maintained for at least 60 m past the ramp intersection. Provision of the "free" right should also be precluded if left-turn movements of any kind are allowed within 125 m of the ramp intersection.

Horizontal sight restrictions may be caused by bridge railings, bridge piers, or slopes. Sight distance is measured between the center of the outside lane approaching the ramp and the eye of the driver of the ramp vehicle assumed 3.0 m back from the edge of shoulder at the crossroads. Figure 504.3J illustrates the determination of ramp setback from an overcrossing structure on the basis of sight distance controlled by the bridge rail. The same relationship exists for sight distance controlled by bridge piers or slopes.

Where ramp set back for the 7-1/2 second criterion is unobtainable, sight distance should be provided by flaring the end of the overcrossing structures or setting back the piers or end slopes of an undercrossing structure.

If signals are warranted within 5 years of construction, consideration may be given to installing signals initially in lieu of providing horizontal sight distance which meets the 7-1/2 second criterion. See Part 4 of the MUTCD and California Supplement, Section 4B.107(CA). However, this is not desirable

and corner sight distance commensurate with design speed should be provided where obtainable (see AASHTO, A Policy on Geometric Design of Highways and Streets).

For additional information on sight distance requirements at signalized intersections, see Index 405.1.

For new construction or major reconstruction of interchanges, the minimum distance (curb return to curb return) between ramp intersections and local road intersections shall be 125 m. The preferred minimum distance should be 160 m. This does not apply to Resurfacing, Restoration and Rehabilitation (RRR), ramp widening, restriping or other projects which do not reconfigure the interchange. This standard does apply to projects proposing to realign a local street.

Where intersections are closely spaced, traffic operations are often inhibited by short weave and storage lengths, and signal phasing. In addition it is difficult to provide proper signing and delineation. Whenever it becomes necessary to locate a ramp terminal close to an intersection, the District Traffic Branch should be consulted regarding the requirement for signing, delineation and signal phasing.

- (4) *Superelevation for Ramps.* The factors controlling superelevation rates discussed in Topic 202 apply also to ramps. As indicated in Table 202.2 use the 12% e_{\max} rate except where snow and ice conditions prevail. In restrictive cases where the length of curve is too short to develop standard superelevation, the highest obtainable rate should be used (see Index 202.5). If feasible, the curve radius can be increased to reduce the standard superelevation rate. Both edge of traveled way and edge of shoulder should be examined at ramp junctions to assure a smooth transition.

Under certain restrictive conditions the standard superelevation rate discussed above may not be required on the curve nearest the ramp intersection of a ramp. The specific conditions under which lower superelevation rates would be considered must be evaluated on a case-by-case basis and must be discussed with the Project Development Coordinator.

Documentation shall be as required by the Coordinator.

- (5) *Single-lane Ramps.* Single lane ramps are those ramps that either enter into or exit from the freeway as a single lane. These ramps are often widened near the ramp intersection with the crossroads to accommodate turning movements onto or from the ramp. When additional lanes are provided near all entrance ramp intersection, the lane drop should be accomplished over a distance equal to $(2/3)WV$. The lane to be dropped should be on the right so that traffic merges left.

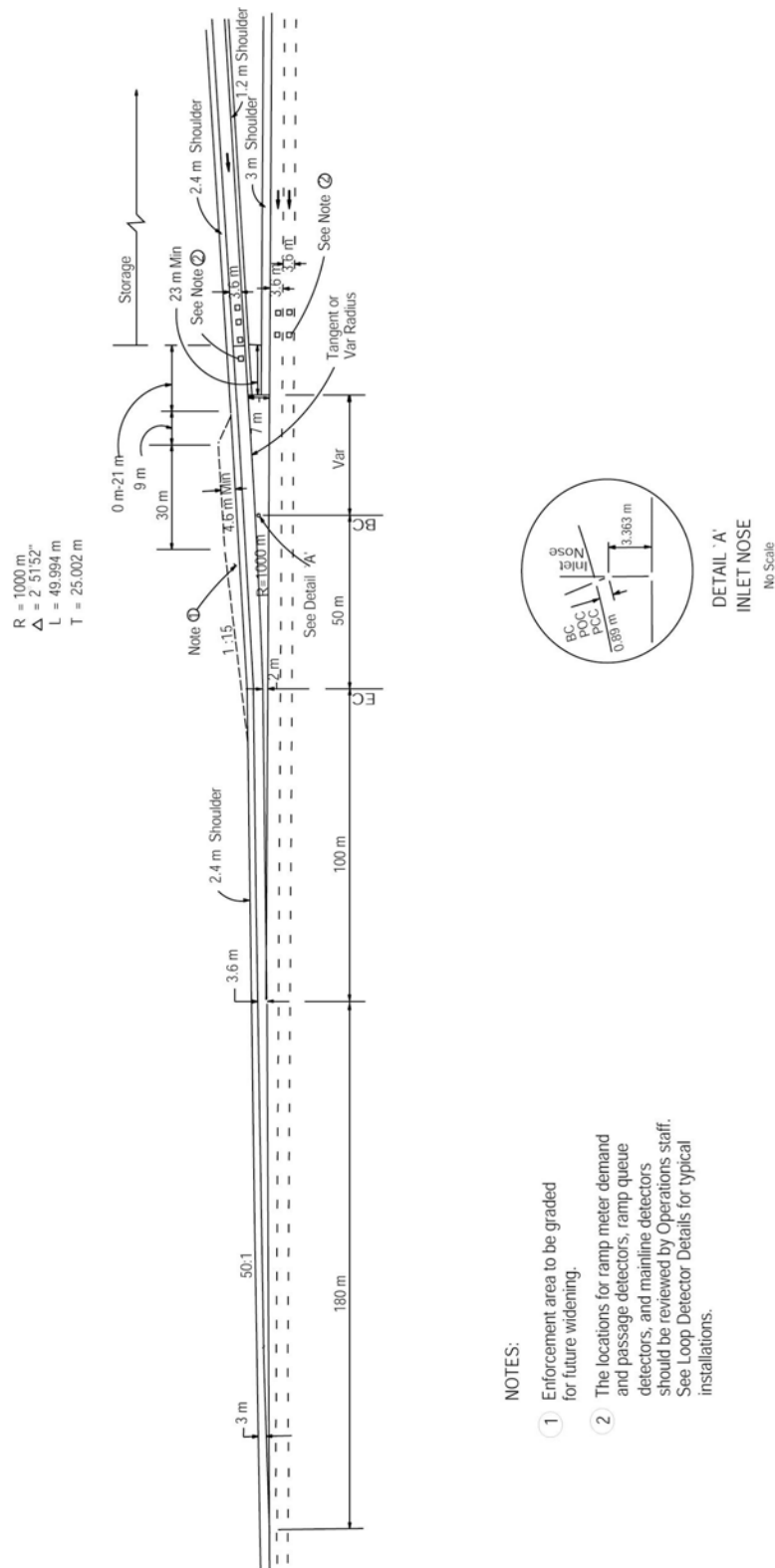
Exit ramps in metropolitan areas may require multiple lanes at the intersection with the crossroads to provide additional storage and capacity. If the length of a single lane ramp exceeds 300 m, an additional lane should be provided on the ramp to permit passing maneuvers. Figure 504.3K illustrates alternative ways of transitioning a single lane exit ramp to two lanes. The decision to use Alternate A or Alternate B is generally based on providing the additional lane for the minor movement.

- (6) *Two-lane Exit Ramps.* Where design year estimated volumes exceed 1500 equivalent passenger cars per hour, a 2-lane width of ramp should be provided initially.

Provisions should be made for possible widening to three or more lanes at the crossroads intersection. Figure 504.3L illustrates the standard design for a 2-lane exit. An auxiliary lane approximately 400 m long should be provided in advance of a 2-lane exit. For volumes less than 1500 but more than 900, a one-lane width exit ramp should be provided with provision for adding an auxiliary lane and an additional lane on the ramp.

- (7) *Two-lane Entrance Ramps.* A standard two lane entrance ramp is illustrated in Figure 504.3L. This design may be utilized in situations where the estimated design year volume exceeds 1500 equivalent passenger cars per hour. The configuration shown in

Figure 504.3A
Typical Freeway Entrance
With 1-Lane Ramp Meter



See the MUTCD and California Supplement for signing and striping typicals.

NOTES:

- See Highway Design Manual Index 504.3 for radii less than 90 m.
- Location for graded enforcement area to be determined by Operations Staff.
- The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors should be reviewed by Operations staff. See Loop Detector Details for typical installations.

**DETAIL 'A'
INLET NOSE**
No Scale

BC
PCC
Inlet
0.60 m
3.363 m

Queue Detector

2.4 m Shoulder

3.6 m

100 m

50 m

2.4 m Shoulder

3.6 m

1.2 m Shoulder

2.4 m Shoulder

7 m

22 m

See Note 3

See Note 2

See Detail 'A'

R=1000 m

2 m

50 m

EC

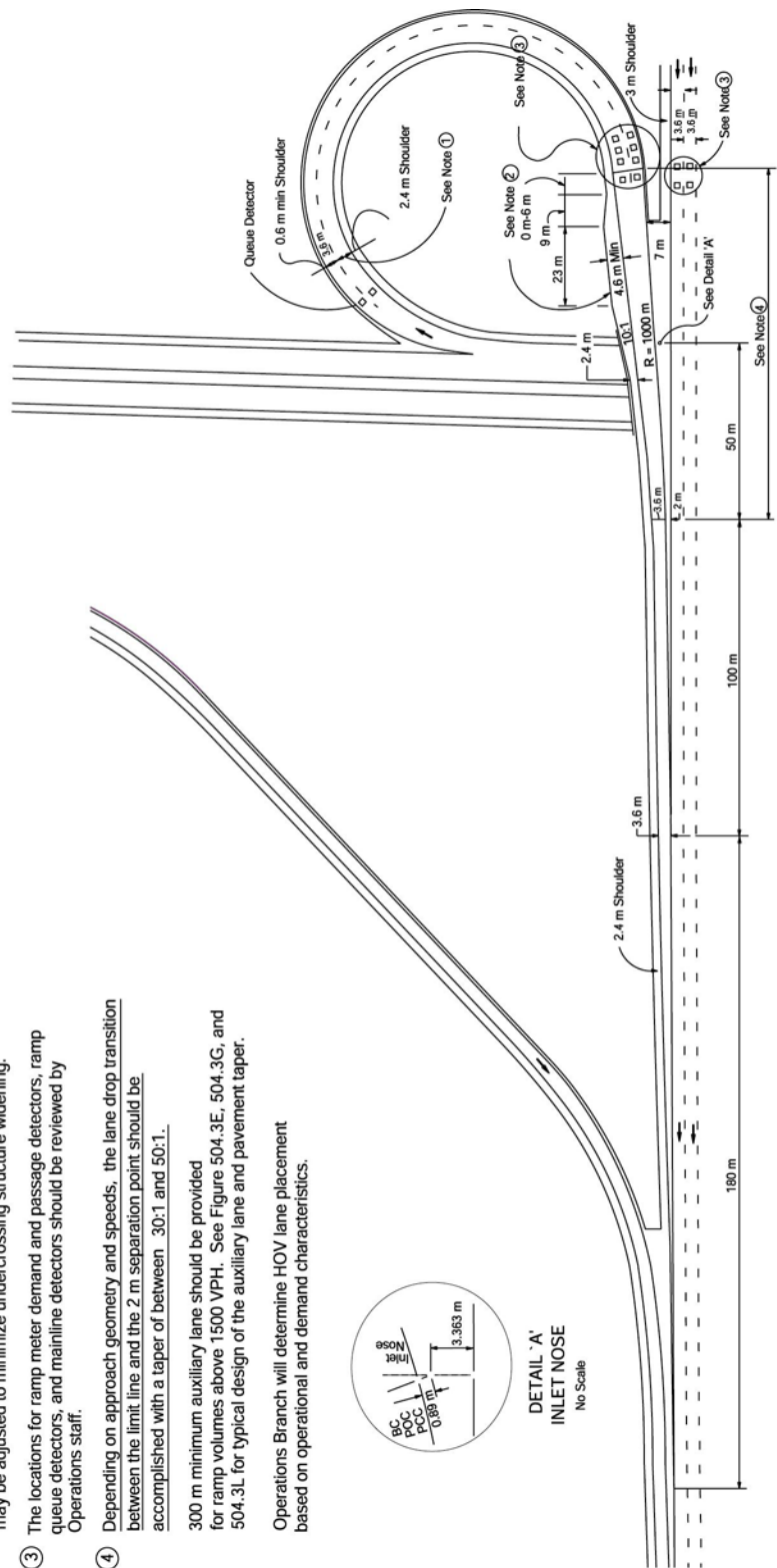
See Note 3

See the MUTCD and California Supplement for signing and striping typicals.

NOTES:

- ① See Highway Design Manual Index 504.3 for radii less than 90 m.
- ② Location for graded enforcement area to be determined by Operations Staff.
- ③ The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors should be reviewed by Operations Staff. See Loop Detector Details for typical installations.

Figure 504.3C
Typical Freeway Entrance Loop Ramp
With 2-Lane Ramp Meter



NOTES:

- ① See Highway Design Manual Index 504.3 for radii less than 90 m.
- ② An enforcement area should be provided when HOV bypass lane is included. Enforcement area dimensions may be adjusted to minimize undercrossing structure widening.
- ③ The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors should be reviewed by Operations Staff.
- ④ Depending on approach geometry and speeds, the lane drop transition between the limit line and the 2 m separation point should be accomplished with a taper of between 30:1 and 50:1.
300 m minimum auxiliary lane should be provided for ramp volumes above 1500 VPH. See Figure 504.3E, 504.3G, and 504.3L for typical design of the auxiliary lane and pavement taper.
Operations Branch will determine HOV lane placement based on operational and demand characteristics.

See the MUTCD and California Supplement for signing and striping typicals.

Figure 504.3D
Typical Freeway Entrance for Ramp Volumes < 1500 VPH
With 2-Lane Ramp Meter

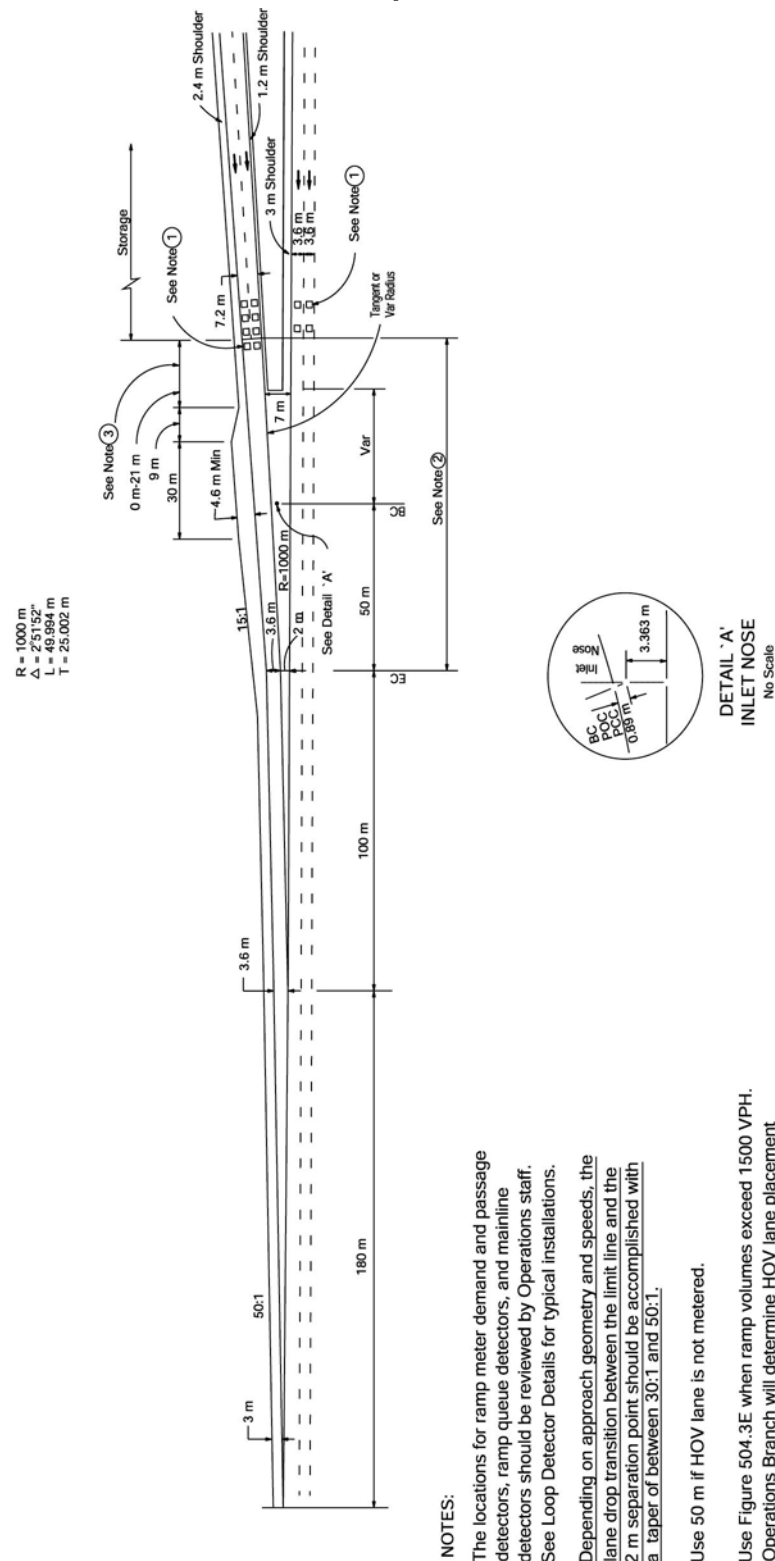


Figure 504.3E
Typical Freeway Entrance for Ramp Volumes > 1500 VPH
With 2-Lane Ramp Meter

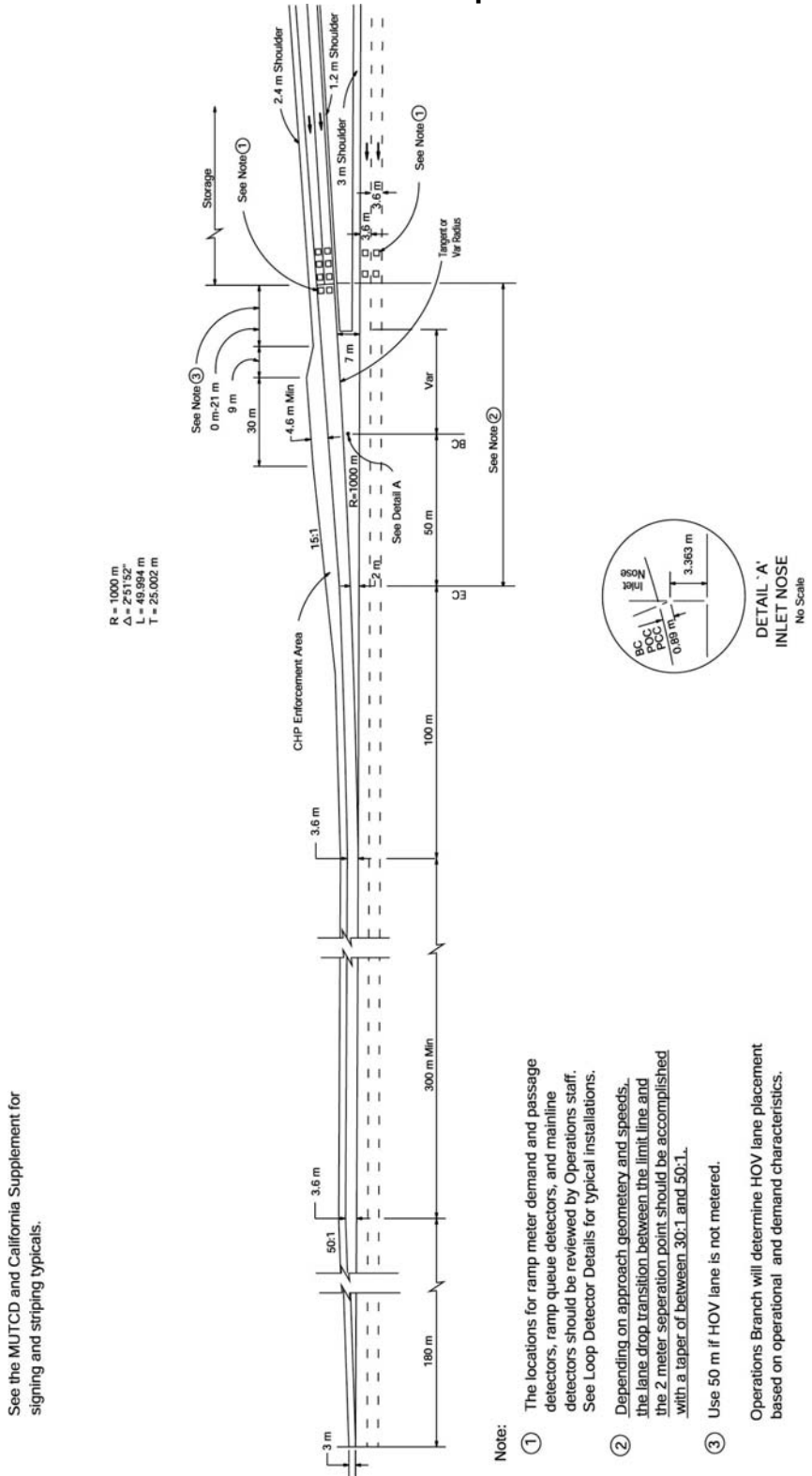
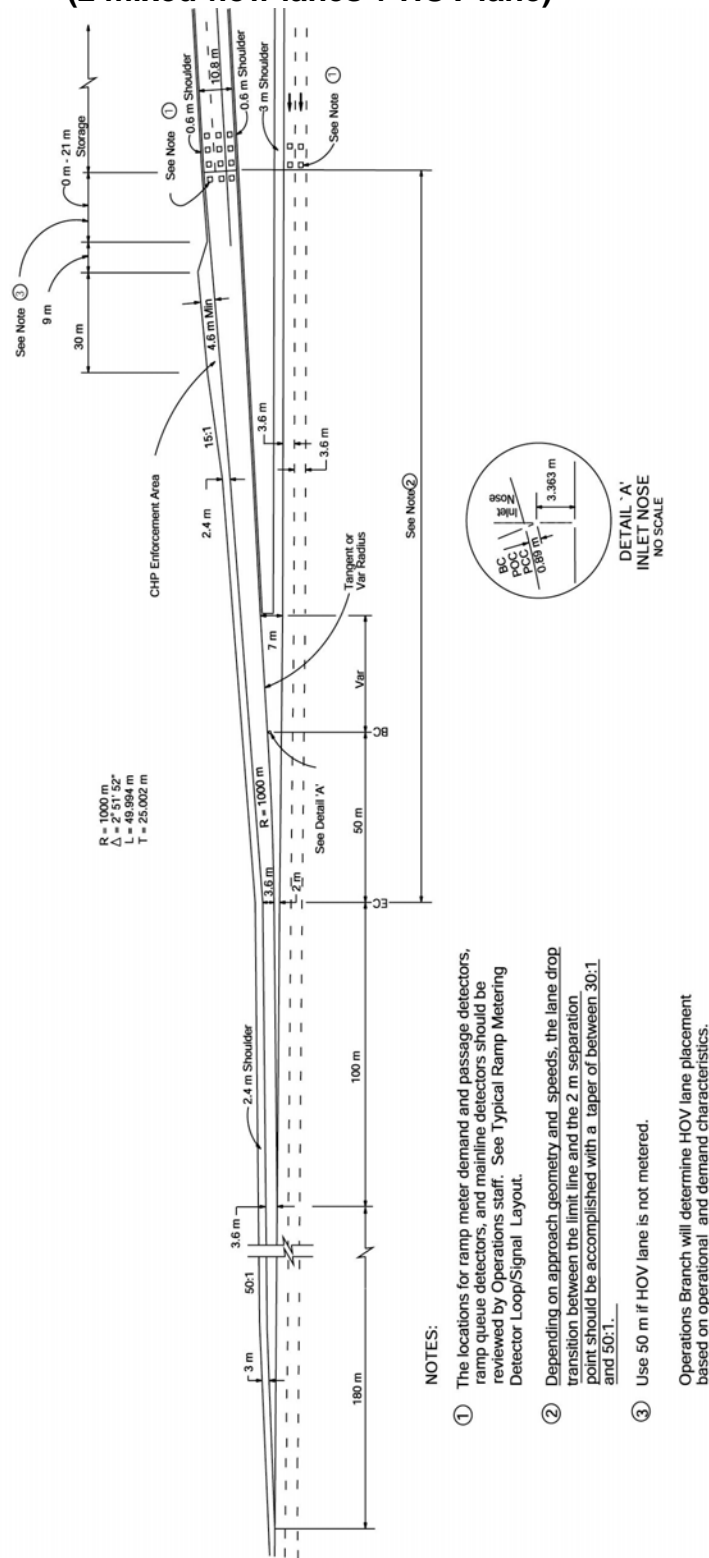


Figure 504.3F
Typical Freeway Entrance for Ramp Volumes < 1500 VPH
3-Lane Ramp Meter
(2 mixed-flow lanes + HOV lane)

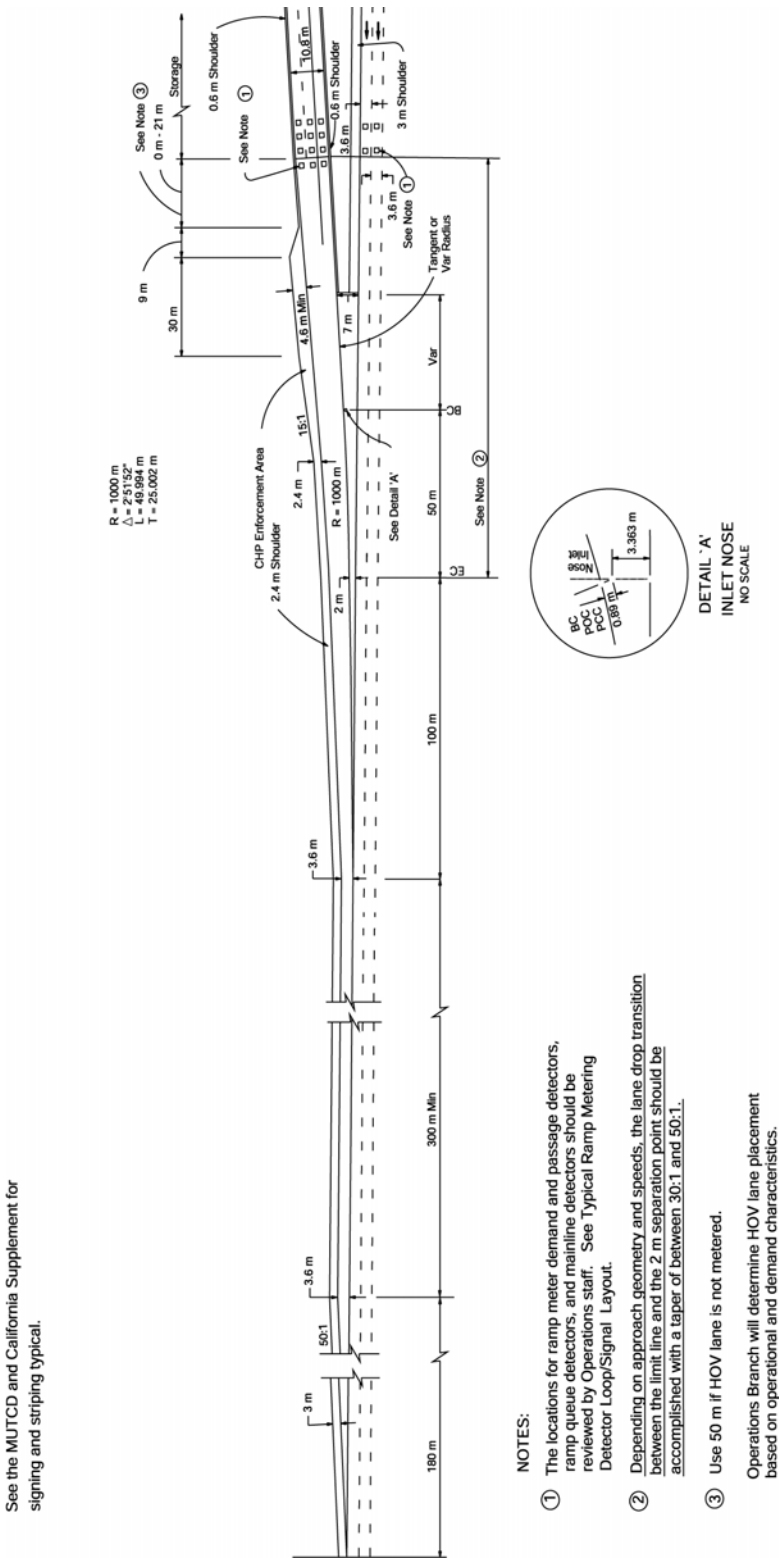
See the MUTCD and California Supplement for signing and striping typicals.



NOTES:

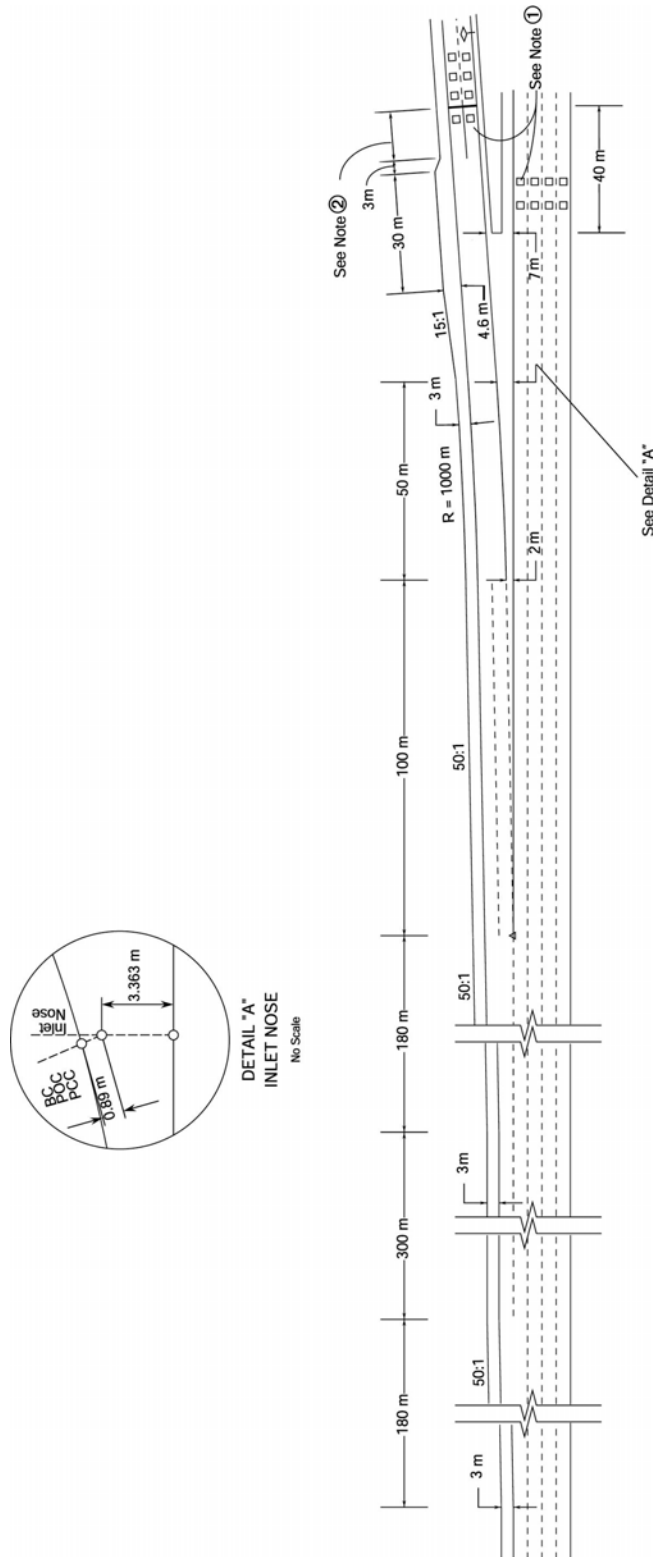
- 1 The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors should be reviewed by Operations staff. See Typical Ramp Metering Detector Loop/Signal Layout.
 - 2 Depending on approach geometry and speeds, the lane drop transition between the limit line and the 2 m separation point should be accomplished with a taper of between 30:1 and 50:1.
 - 3 Use 50 m if HOV lane is not metered.
- Operations Branch will determine HOV lane placement based on operational and demand characteristics.

Figure 504.3G
Typical Freeway Entrance for Ramp Volumes > 1500 VPH
3-Lane Ramp Meter
(2 mixed-flow lanes + HOV lane)



- NOTES:
- 1 The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors should be reviewed by Operations staff. See Typical Ramp Metering Detector Loop/Signal Layout.
 - 2 Depending on approach geometry and speeds, the lane drop transition between the limit line and the 2 m separation point should be accomplished with a taper of between 30:1 and 50:1.
 - 3 Use 50 m if HOV lane is not metered.
- Operations Branch will determine HOV lane placement based on operational and demand characteristics.

Figure 504.3H
Typical Freeway Connector
2-Lane Meter
(1 mixed-flow lane + HOV lane)



NOTES:

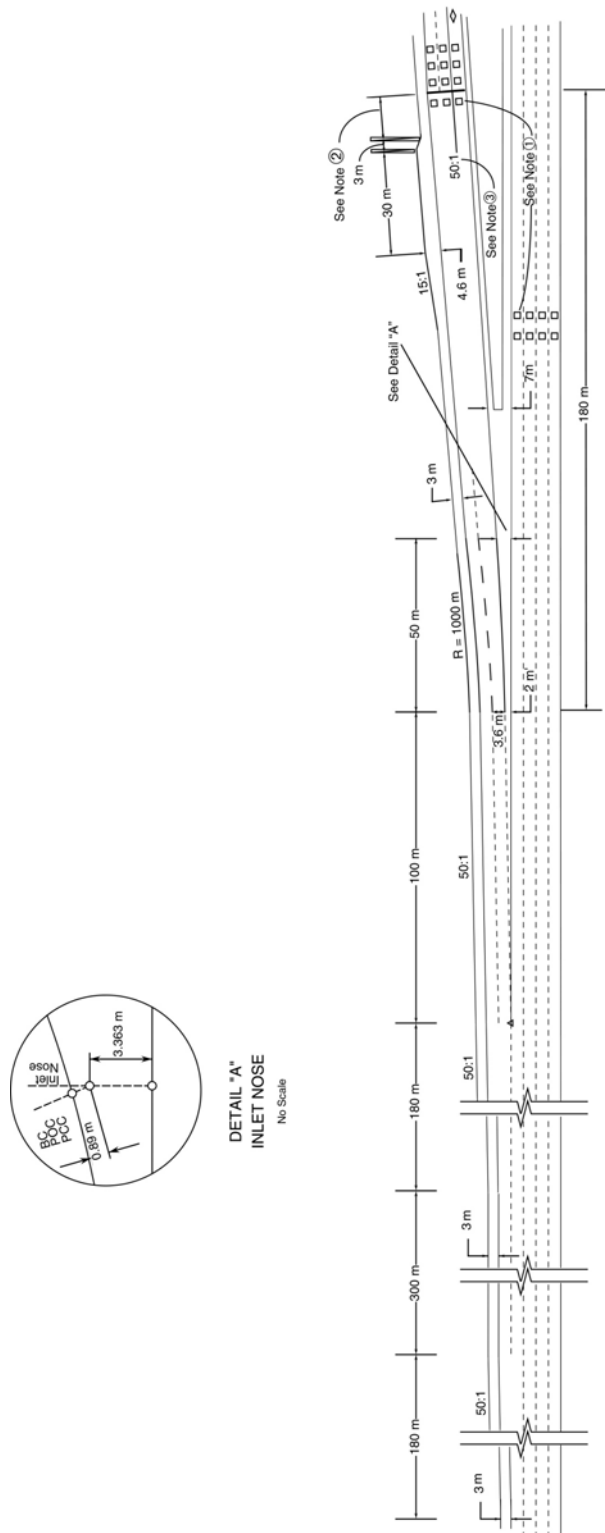
① The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors should be reviewed by Operations staff. See Typical Ramp Metering Detector Loop/Signal Layout.

② Use 0 m–21 m if HOV lane is metered. Use 50 m if HOV lane is not metered.

Operations staff will determine HOV lane placement based on operational and demand characteristics.

Typically, lane drops are to be accomplished over a distance equal to 2/3 WV, but the lane drop transition should be accomplished with at least a 50:1 taper.

Figure 504.3I
Typical Freeway Connector
3-Lane Meter
(2 mixed-flow lanes + HOV lane)

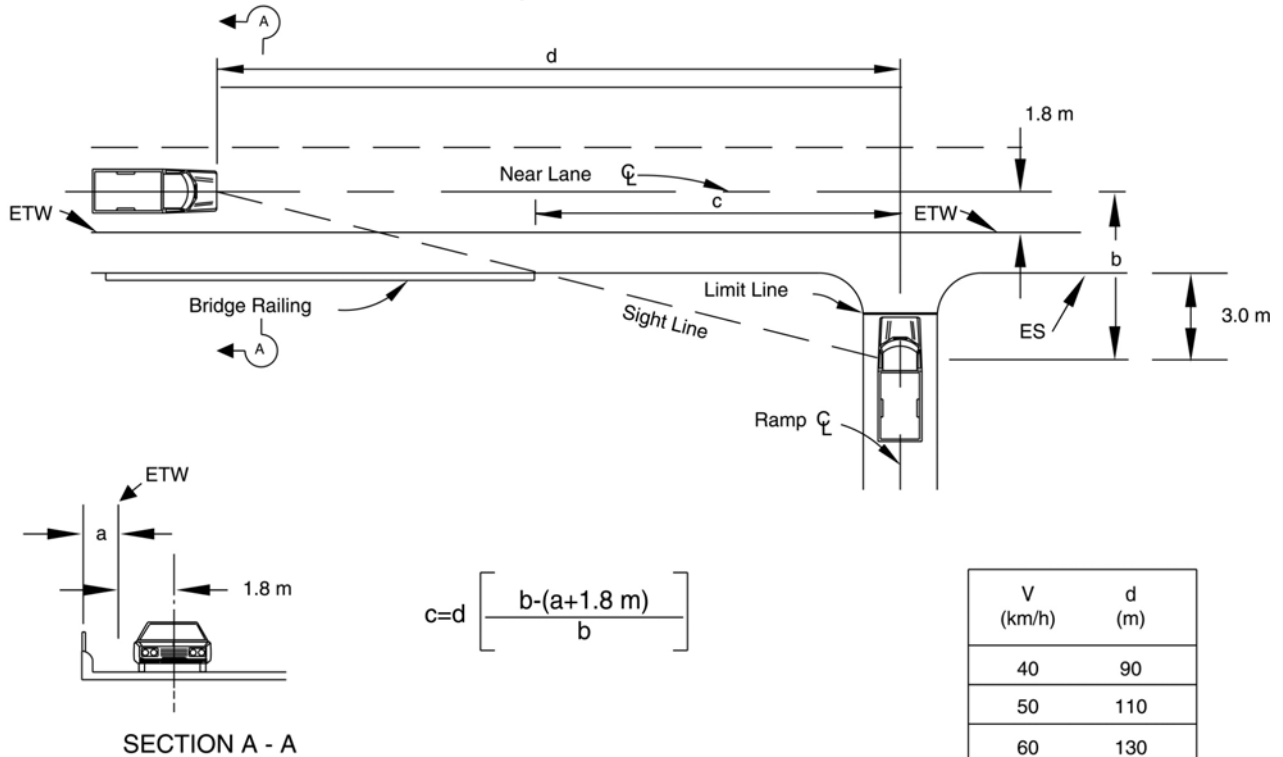


NOTES:

- ① The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors should be reviewed by Operations staff. See Typical Ramp Metering Detector Loop/Signal Layout.
- ② Use 0 m–21 m if HOV lane is metered. Use 50 m if HOV lane is not metered.
- ③ Typically, lane drops are to be accomplished over a distance equal to $2/3$ WV, but the lane drop transition should be accomplished with at least a 50:1 taper. Operations staff will determine HOV lane placement based on operational and demand characteristics.

Figure 504.3J
Location of Ramp Intersections
on the Crossroads

Unsignalized and based on 7.5 second
horizontal sight distance criteria



a=Distance from edge of traveled way to bridge railing.

b=Distance from center of near lane to eye of ramp vehicle driver.

Ramp driver's eye is assumed to be located 3 m from the edge of shoulder, but not less than 4 m from the ETW. (Therefore, b = 1.8 m + shoulder width + 3.0 m) See Index 405.1.

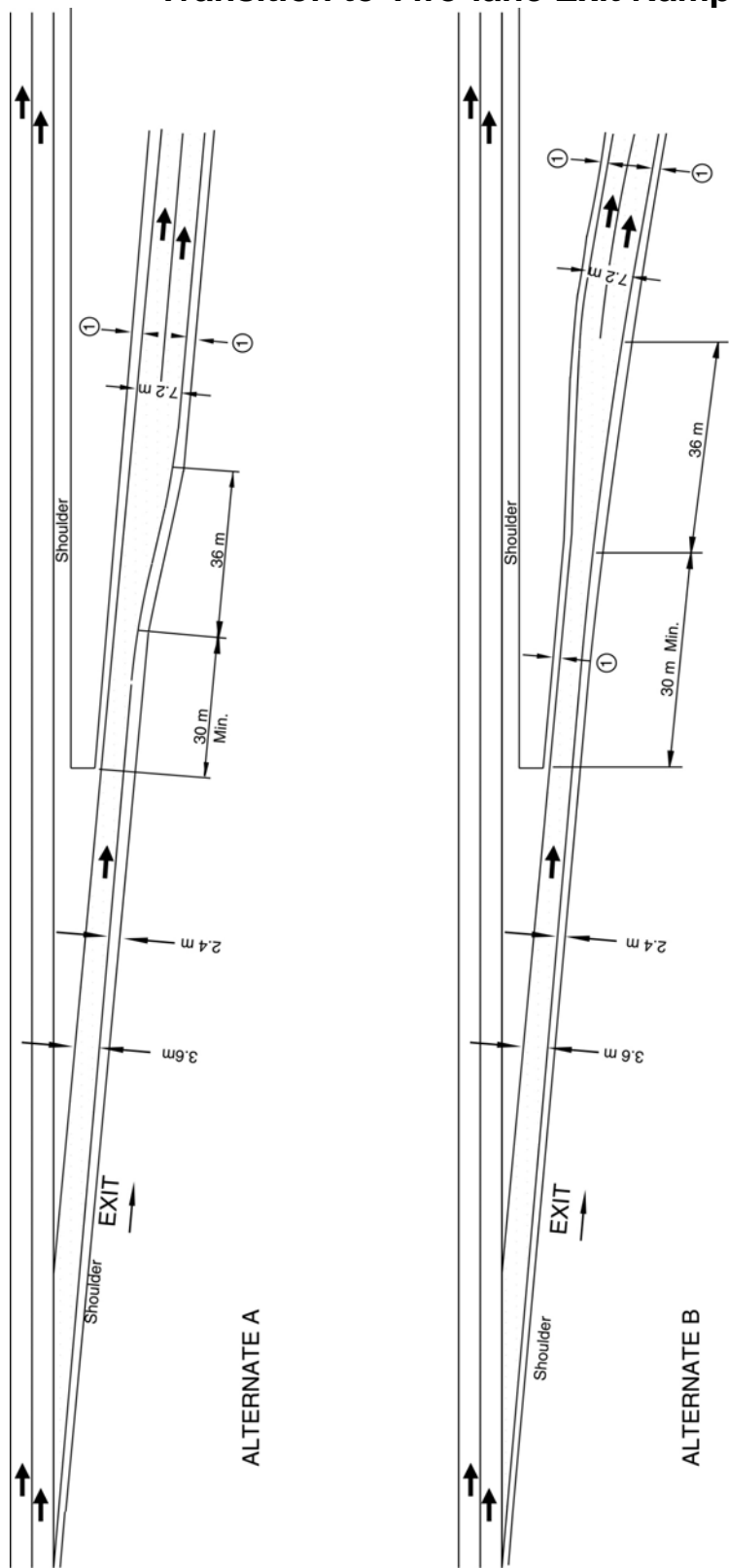
c=Ramp set back from end of bridge railing.

d=Corner Sight distance along highway from intersection. (See Table above.)

Sight distance is measured from a 1070 mm eye height on the ramp to a 1300 mm object height on the crossroad.

V=Anticipated prevailing speed on crossroad.

Figure 504.3K
Transition to Two-lane Exit Ramp



NOTES:

- ① See Table 302.1 for shoulder widths. If shoulder reductions occur, see index 206.3(4) for transitions.

[illegible]

Figure 504.3L, which includes the provision of a 300 m auxiliary lane parallel to the freeway, will typically only be used where adequate capacity exists on the effected corridor of the through facility in the design year. Where capacity is limited, consideration should be given to extending the auxiliary lane to the next interchange or adding additional lanes to the freeway. For most situations, the multiple ramp lanes taper to a single lane prior to the 2-meter separation point (where merging is considered to begin). A thorough investigation of ramp volumes versus through facility volumes must be made for off-peak as well as peak periods if metering of the ramp is anticipated. Early discussion with the HQ Traffic Liaison and Design Coordinator or Design Reviewer is recommended whenever two-lane entrance ramps are being considered.

- (8) *Loop Ramps.* Normally, loop ramps should have one lane and shoulders unless a second lane is needed for capacity or ramp metering purposes. Consideration should be given to providing a directional ramp when loop volumes exceed 1500 vehicles per hour. If two lanes are provided, normally only the right lane needs to be widened for trucks. See Topic 404 for additional discussion on lane widths and design of ramp intersections to accommodate the design vehicle. See Index 504.3(1) for a discussion on ramp widening for trucks.

Radii for loop ramps should normally range from 45 m to 60 m. Increasing the radii beyond 60 m is typically not cost effective as the slight increase in design speed is usually outweighed by the increased right of way requirements and the increased travel distance. Curve radii of less than 35 m should also be avoided. Extremely tight curves lead to increased off-tracking by trucks and increase the potential for vehicles to enter the curve with excessive speed.

Of particular concern in the design of loop ramps are the constraints imposed on large trucks. Research indicates that trucks often enter loops with excessive speed, either due to inadequate deceleration on exit ramps or due

to driver efforts to maintain speed on entrance ramps to facilitate acceleration and merging. Where the loop is of short radius and is also on a steep descent (over 6%), it is important to develop the standard 2/3 full superelevation rate by the beginning of the curve (see Index 504.2(5)). On loop entrance ramps this can often be facilitated by beginning the ramp with a short tangent (20 m to 30 m) that diverges from the cross street at an angle of 4 to 9 degrees. Consideration should be given to developing additional tangent length if conditions allow.

The ramp lane pavement structure should be provided on shoulders for curves with a radius less than 90 m (see Indexes 626.1 and 636.1).

- (9) *Distance Between Successive On-ramps.* The minimum distance between two successive on-ramps to a freeway lane should be the distance needed to provide the standard on-ramp acceleration taper shown on Figure 504.2A. This distance should be about 300 m unless the upstream ramp adds an auxiliary lane in which case the downstream ramp should merge with the auxiliary lane in a standard 50:1 convergence. The distance between on-ramp noses will then be controlled by interchange geometry.
- (10) *Distance Between Successive Exits.* The minimum distance between successive exit ramps for guide signing should be 300 m on the freeway and 180 m on collector-distributor roads.
- (11) *Curbs.* Curbs should not be used on ramps except in the following locations:
- A Type B-100 or Type D curb (see Index 303.2) may be used on both sides of the separation between freeway lanes and a parallel collector-distributor road.
 - A B4 curb may be used as shown in Figure 504.2A to control drainage or where the gore cross slope would be greater than allowed in Index 504.2(5). When the optional B4 curb is used at the entrance ramp inlet nose, the shoulder adjacent to the curb should be the same width as the ramp shoulder approaching

the curb. The B4 gutter pan can be included as part of the shoulder width. As stated in Index 405.4(2), curbs are typically discouraged where design speeds are over 75 km/h. The appropriateness of curbs at gore areas must be determined on a case-by-case basis.

- (c) Curbs may be used where necessary at the ramp connection with the local street for the protection of pedestrians, for channelization, and to provide compatibility with the local facility.
- (d) The Type E curb may be used only in special drainage situations, for example, where drainage parallels and flows against the face of a retaining wall.

In general, curbs should not be used on the high side of ramps or in off-ramp gore areas except at collector-distributor roads. The offtracking of trucks should be analyzed when considering curbs on ramps.

- (12) *Dikes.* Dikes may be used where necessary to control drainage. For additional information see Index 303.3.

504.4 Freeway-to-Freeway Connections

- (1) *General.* All of the design criteria discussed in Indexes 501.3, 504.2 and 504.3 apply to freeway to freeway connectors, except as discussed or modified below.
- (2) *Design Speed.* The design speed for single lane directional and all branch connections should be a minimum of 80 km/h. When smaller radius curves, with lower design speeds, are used the vertical sight distance should be consistent with approaching vehicle speeds. Design speed for loop connectors should be consistent with Index 504.2(4).
- (3) *Grades.* The maximum profile grade on freeway-to-freeway connections should not exceed 6%. Flatter grades and longer vertical curves than those used on ramps are needed to obtain increased stopping sight distance for higher design speeds.

(4) Shoulder Width.

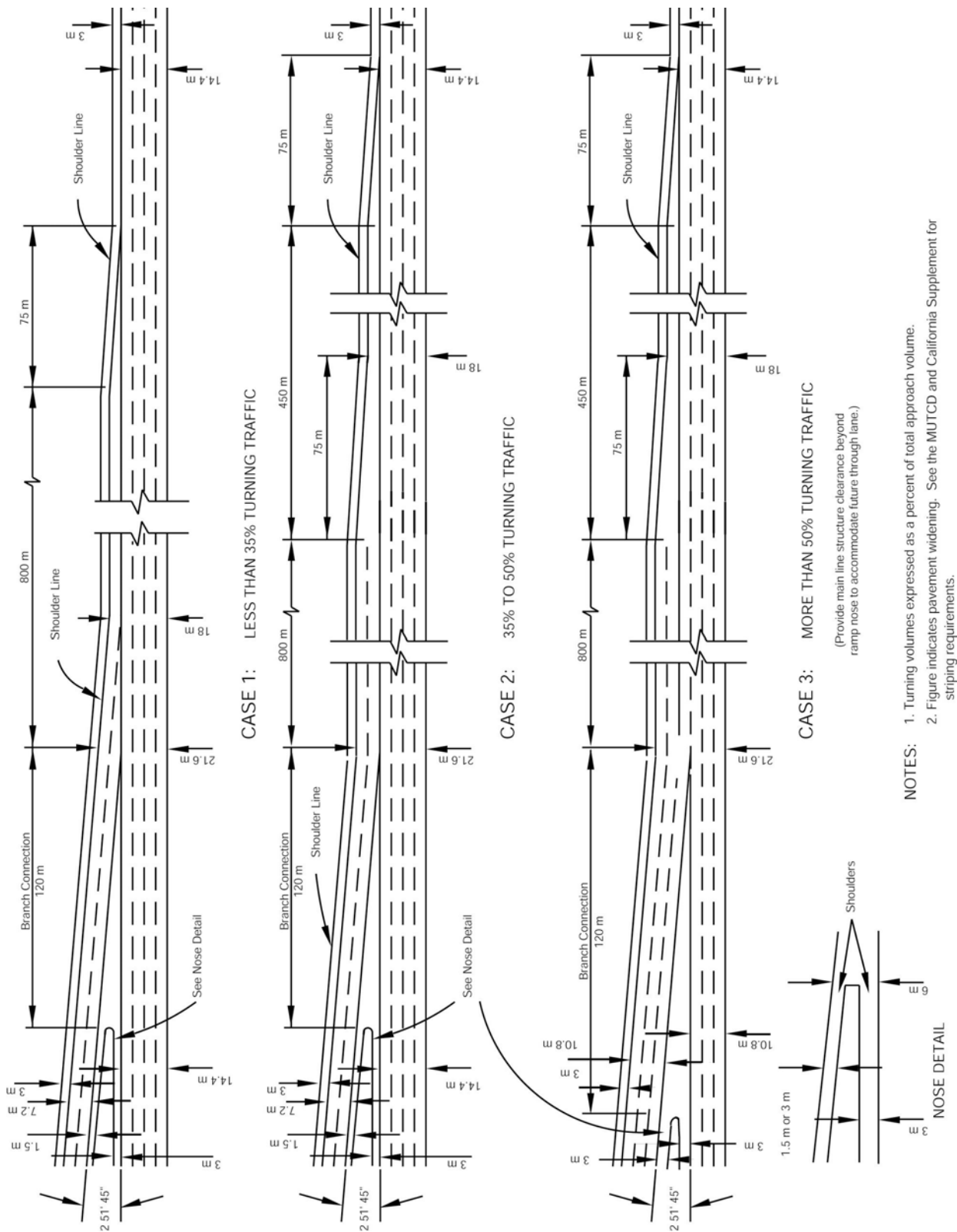
- (a) Single-lane and Two-lane Connections--**The width of shoulders on single-lane and two-lane (except as described below) freeway-to-freeway connectors shall be 1.5 m on the left and 3.0 m on the right. A single lane freeway-to-freeway connector that has been widened to two lanes solely to provide passing opportunities and not due to capacity requirements shall have a 1.5 m left shoulder and at least a 1.5 m right shoulder** (see Index 504.4(5)).
- (b) Three-lane Connections--**The width of shoulders on three-lane connectors shall be 3.0 m on both the left and right sides.**

- (5) *Single-lane Connections.* Freeway-to-freeway connectors may be single lane or multilane. Where design year volume is between 900 and 1500 equivalent passenger cars per hour, initial construction should provide a single lane connection with the capability of adding an additional lane. Single lane directional connectors should be designed using the general configurations shown on Figure 504.2A and 504.2B, but utilizing the flatter divergence angle shown in Figure 504.4. Single lane loop connectors may use a diverge angle of as much as that shown on Figure 504.2B for ramps, if necessary. The choice will depend upon interchange configuration and driver expectancy. Single lane connectors in excess of 300 m in length should be widened to two lanes to provide for passing maneuvers (see Index 504.4(4)).

- (6) *Branch Connections.* A branch connection is defined as a multilane connection between two freeways. A branch connection should be provided when the design year volume exceeds 1500 equivalent passenger cars per hour.

Merging branch connections should be designed as shown in Figure 504.3L. Diverging branch connections should be designed as shown in Figure 504.4. The diverging branch connection leaves the main

Figure 504.4 Diverging Branch Connections



NOTES:

1. Turning volumes expressed as a percent of total approach volume.
2. Figure indicates pavement widening. See the MUTCD and California Supplement for striping requirements.

freeway lanes on a flatter angle shown in Figure 504.4 than the standard 2-lane ramp exit connection shown in Figure 504.3K. The standard ramp exit connects to a local street. The diverging branch connection connects to another freeway and has a flatter angle that allows a higher departure speed.

At a branch merge, an 800 m length of auxiliary lane should be provided beyond the merge of one lane of the inlet, except where it does not appear that capacity on the freeway will be reached until five or more years after the 20 year design period. In this case the length of auxiliary lane should be a minimum of 300 m. For diverging connections where less than capacity conditions beyond the design year are anticipated, the length of auxiliary lane in advance of the exit should be 400 m.

- (7) *Lane Drops.* The lane drop taper on a freeway-to-freeway connector should not be less than (2/3)WV.
- (8) *Metering.* Any decision to meter freeway-to-freeway connectors must be carefully considered as driver expectancy on these types of facilities is for high-speed uninterrupted flow. If metering is anticipated on a connector, discussions with the HQ Traffic Liaison and Design Coordinator should take place as early as possible. Issues of particular concern are adequate deceleration lengths to the end of the queue, potential need to widen shoulders if sight distance is restricted (particularly on ramps with 1.5 m shoulders on each side), and the potential for queuing back onto the freeway.

504.5 Auxiliary Lanes

In order to ensure satisfactory operating conditions, auxiliary lanes may be added to the basic width of traveled way.

Where an entrance ramp of one interchange is closely followed by an exit ramp of another interchange, the acceleration and deceleration lanes should be joined with an auxiliary lane. Auxiliary lanes should be provided in all cases when the weaving distance, measured as shown in Figure 504.2A, is less than 600 m. Where interchanges

are more widely spaced and ramp volumes are high, the need for an auxiliary lane between the interchanges should be determined in accordance with Index 504.7.

Auxiliary lanes may be used for the orientation of traffic at 2-lane ramps or branch connections as illustrated on Figure 504.3L and Figure 504.4. The length and number of auxiliary lanes in advance of 2-lane exits are based on percentages of turning traffic and a weaving analysis.

Auxiliary lanes should be considered on all freeway entrance ramps with significant truck volumes. The grade, volumes and speeds should be analyzed to determine the need for auxiliary lanes. An auxiliary lane would allow entrance ramp traffic to accelerate to a higher speed before merging with mainline traffic, or simply provide more opportunity to merge. See Index 504.2 for specific requirements.

504.6 Mainline Lane Reduction at Interchanges

The basic number of mainline lanes should not be dropped through a local service interchange. The same standard should also be applied to freeway-to-freeway interchanges where less than 35% of the traffic is turning (see Figure 504.4). Where more than 35% of the freeway traffic is turning, consideration may be given to reducing the number of lanes. No decision to reduce the number of lanes should be made without the approval of the District Traffic Operations Unit. Additionally, adequate structure clearance (both horizontal and vertical) should be provided to accommodate future construction of the dropped lane if required.

Where the reduction in traffic volumes is sufficient to warrant a decrease in the basic number of lanes, a preferred location for the lane drop is beyond the influence of an interchange and preferably at least 1 km from the nearest exit or inlet nose. It is desirable to drop the right lane on tangent alignment with a straight or sag profile so vehicles can merge left with good visibility to the pavement markings in the merge area (see Index 201.7).

504.7 Weaving Sections

A weaving section is a length of one-way roadway where vehicles are crossing paths, changing lanes, or merging with through traffic as they enter or exit a freeway or collector-distributor road.

A single weaving section has an inlet at the upstream end and an exit at the downstream end. A multiple weaving section is characterized by more than one point of entry followed by one or more points of exit.

A rough approximation for adequate length of a weaving section is 0.3 m of length per weaving vehicle per hour. This rate will approximately provide a level of service C. Refer to the January 31, 1995 Design Information Bulletin Number 77 on Interchange Spacing for additional weaving requirements.

There are various methods for analyzing weaving sections. Two methods which provide valid results are described below.

The Leisch method, which is usually considered the easiest to use, is illustrated in Figure 504.7A. This method was developed by Jack Leisch & Associates and may be used to determine the length of weaving sections for both freeways and collector-distributor roads. The Leisch weaving charts determine the level of service for the weaving volumes for the length of the weaving section from the first panel on the lower left of the chart. The analysis is dependent on whether the section is balanced or unbalanced, as defined in Figure 504.7B. The level of service for the total volume over all lanes of the weaving section is then found from the panels on the right of the chart. The weaving chart should not be extrapolated.

Pages 234-238 of the 1965 Highway Capacity Manual (HCM) provide a method for determining the adequacy of weaving sections near single lane ramps. It is often referred to as the Level of Service (LOS) D method. This method is also documented in Traffic Bulletin 4 which is available from the District Division of Traffic Operations. The LOS D method can be used to project volumes along a weaving section. These volumes can be compared to the capacities along the same weaving section.

Volumes in passenger car equivalents per hour (PCEPH) should be adjusted for freeway grade and truck volumes. Table 504.7C and Figures 504.7D and E are reprinted from the 1965 HCM and provide information regarding vehicle distribution by lane.

The results obtained from Figure 504.7A (the Leisch Method) for single-lane ramps with an auxiliary lane and weaving rates exceeding 2500 PCEPH should be checked using the LOS D method.

Weaving capacity analyses other than those described above should not be used on California highways. Other methods, such as the one contained in the 1994 HCM, may not always produce accurate results.

Weaving sections in urban areas should be designed for level of service C or D. Weaving sections in rural areas should be designed for level of service B or C. Design rates for lane balanced weaving sections where at least one ramp or connector will be two lanes should not result in a level of service lower than the middle of level of service D using Figure 504.7A. In determining acceptable hourly operating volumes, peak hour factors should be used.

On main freeway lanes the weaving length measured as shown in Figure 504.2A should not be less than 500 m except where excessive cost or severe environmental constraints would require consideration of a shorter length. 300 m of length should be added for each additional lane to be crossed by weaving vehicles. The volumes used shall be volumes unconstrained by metering regardless of whether metering will be used. It should be noted that a weaving analysis must be considered over an entire freeway segment as weaving can be affected by other nearby ramps.

The District Traffic Operations Branch should be consulted for difficult weaving analysis problems.

504.8 Access Control

Access rights shall be acquired along interchange ramps to their junction with the nearest public road. At such junctions, for new construction, access control should extend 30 m beyond the end of the curb return or ramp radius in

urban areas and 100 m in rural areas, or as far as necessary to ensure that entry onto the facility does not impair operational characteristics. **Access control shall extend at least 15 m beyond the end of the curb return, ramp radius, or taper.**

Typical examples of access control at interchanges are shown in Figure 504.8. These illustrations do not presume to cover all situations or to indicate the most desirable designs for all cases. When there is state-owned access control on both sides of a local road, a maintenance agreement may be needed.

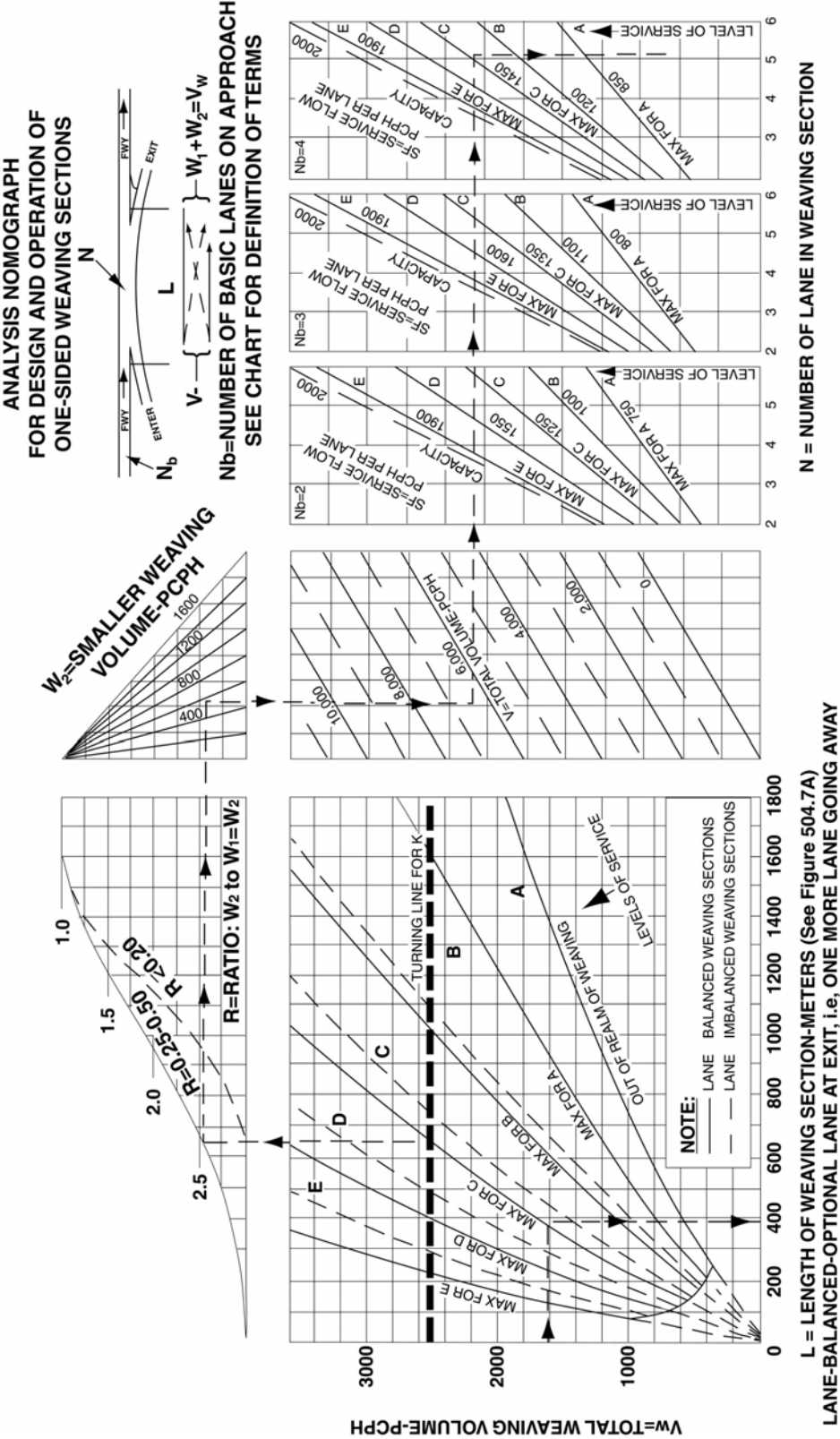
For new construction or major reconstruction, access rights should be acquired on the opposite side of the local road from ramp terminals to preclude the construction of future driveways or local roads within the ramp intersection. This access control would limit the volume of traffic and the number of phases at the intersection of the ramp and local facility, thereby optimizing capacity and operation of the ramp. Through a combination of access control and the use of raised median islands along the local facility, intersections should be located at least 125 m from the ramp intersection. Right in - right out access may be permitted beyond 60 m from the ramp intersection. The length of access control on both sides of the local facility should match.

In Case 2 consider private ownership within the loop only if access to the property is an adequate distance from the ramp junction to preserve operational integrity.

In Case 3 if the crossroads is near the ramp junction at the local road, full access control should be acquired on the local road from the junction to the intersection with the crossroad.

Case 6 represents a slip ramp design. If the ramp is perpendicular to the local/frontage road refer to Case 3. In Case 6 if the crossroad is near the ramp junction to the local/frontage road, access control should be acquired on the opposite side of the local road from the junction.

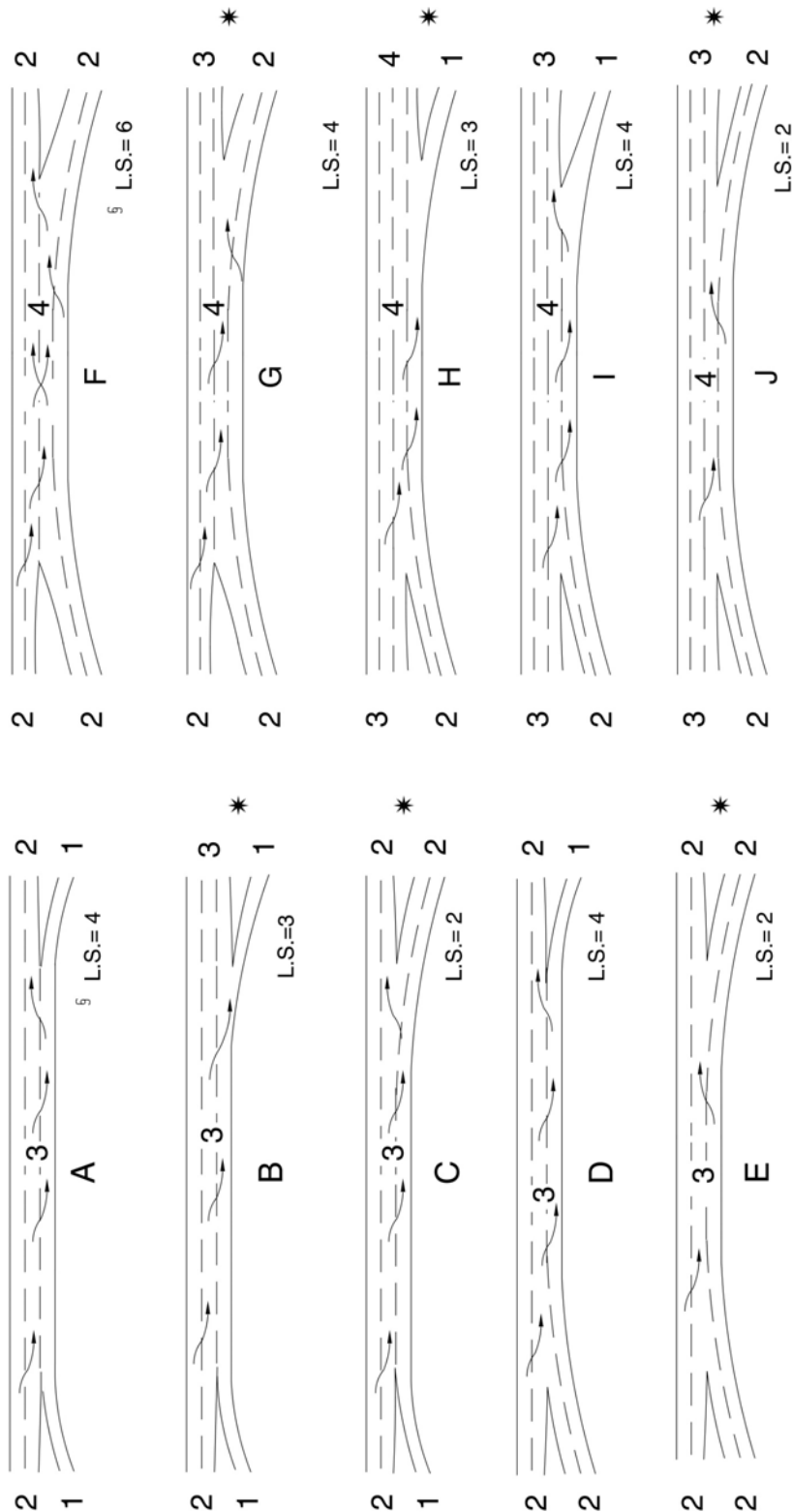
Figure 504.7A
Design Curve for Freeway and Collector Weaving



NOTE: EXTRAPOLATION OF CHART BEYOND THE BOUNDARIES GIVEN IS NOT ADVISED.

Example: The nomograph is entered on the left (see dashed line and arrows) with weaving volume, $W_1 + W_2$ (or V_w) followed by projection to the right, intersecting the desired weaving LOS; a vertical drop from this point provides weaving distance $L = 400$ m. Returning to first intersection point of V_w with LOS line, an upward projection along the LOS is intersected with the horizontal, heavy dashed, "turning line for K" from here the solution line is extended vertically to intersect the K values curve, from which a horizontal extension meets the desired W_2 volume. Then a downward turn to total volume, V , from which the line is horizontally projected to the right, intersection (in this case) the desired LOS = C curve having an SF of 1450 (representing the overall or composite operation of the weaving section), from which a downward extension yields a N of 5.2: this would be rounded to $N = 5$ lanes.

Figure 504.7B
Lane Configuration of Weaving Sections



Source: Jack E. Leisch & Associates

* DENOTE LANE BALANCE - OPTIONAL LANE AT EXIT

[§] L.S. - POTENTIAL LANE SHIFTS, CONSIDERING MAX. OF 2 LANES INVOLVED ON ANY ONE APPROACH

Table 504.7C
Percent of Through Traffic Remaining in Outer Through Lane
(Level of Service D Procedure)

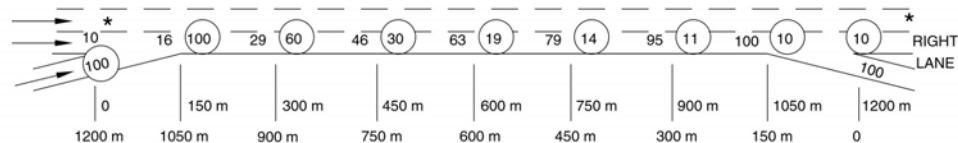
TOTAL VOLUME OF THROUGH TRAFFIC, ONE DIRECTION (vph)	APPROXIMATE PERCENTAGE OF THROUGH ^a TRAFFIC REMAINING IN THE OUTER THROUGH LANE IN THE VICINITY OF RAMP TERMINALS AT LEVEL OF SERVICE D.		
	8-LANE ^b FREEWAY	6-LANE ^c FREEWAY	4-LANE ^d FREEWAY
6500 and over	10	-	-
6000 - 6499	10	-	-
5500 - 5999	10	-	-
5000 - 5499	9	-	-
4500 - 4999	9	18	-
4000 - 4499	8	14	-
3500 - 3999	8	10	-
3000 - 3499	8	6	40
2500 - 2999	8	6	35
2000 - 2499	8	6	30
1500 - 1999	8	6	25
Up to 1499	8	6	20

- a. Traffic not involved in a ramp movement within 1200 m in either direction.
- b. 4 lanes one way
- c. 3 lanes one way
- d. 2 lanes one way

Figure 504.7D Percentage Distribution of On- and Off-ramp Traffic in Outer Through Lane and Auxiliary Lane (Level of Service D Procedure)

CASE I - SINGLE - LANE ON- AND OFF-RAMPS WITHOUT AUXILIARY LANE

(THIS CHART MAY BE USED REGARDLESS OF ACTUAL SPACING BETWEEN ON- AND OFF-RAMPS, BUT AS NOTED BELOW* CAUTION MUST BE EXERCISED IN USING THESE VALUES.)



CASE II - SINGLE - LANE - ON- AND OFF-RAMPS WITH AUXILIARY LANE**

(A) L = 300 m

EXAMPLE:

GIVEN: L = 300 m

PORTION OF V THROUGH

(FROM TABLE 504.7C = 475 VPH

ON-RAMP = 1,000 VPH

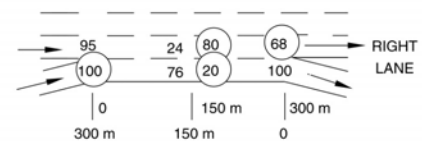
OFF-RAMP = 1,200 VPH

ON-RAMP TO OFF-RAMP = 0

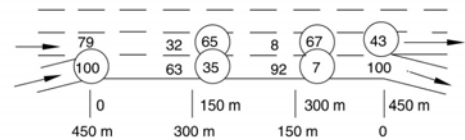
FIND: V (VOL. IN OUTER THROUGH LANE) @ 150 m =

$$475 + (0.80)(1,000) + (0.24)(1,200) =$$

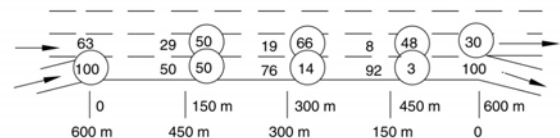
$$1,563 \text{ VPH}$$



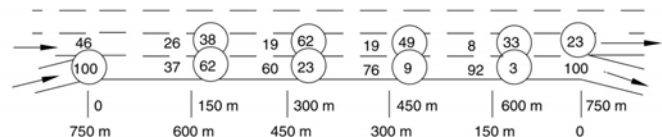
(B) L = 450 m



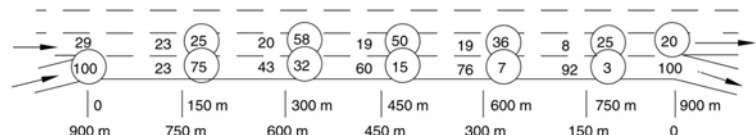
(C) L = 600 m



(D) L = 750 m



(E) L = 900 m



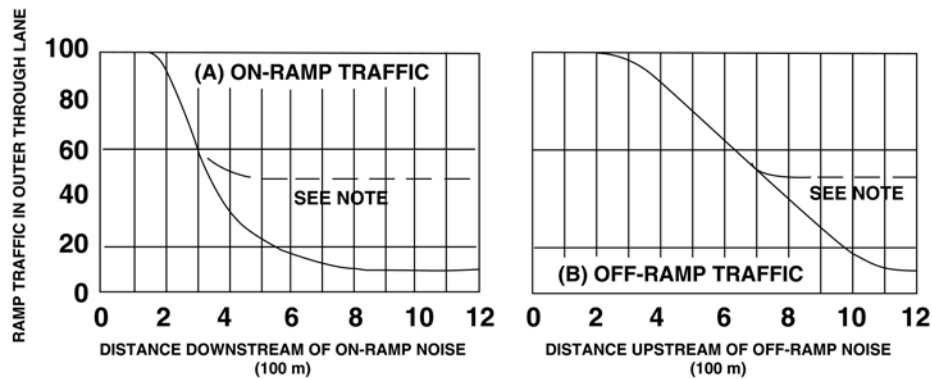
CIRCLED VALUES INDICATE PERCENTAGE OF ON-RAMP TRAFFIC IN LANE SHOWN. UNCIRCLED VALUES INDICATE PERCENTAGE OF OFF-RAMP TRAFFIC IN LANE SHOWN. (REMAINING PORTION OF TRAFFIC IS IN LANE(S) TO LEFT OF OUTER THROUGH LANE.)

THESE PERCENTAGES ARE NOT NECESSARILY THE DISTRIBUTIONS UNDER FREE FLOW OR LIGHT RAMP TRAFFIC, BUT UNDER PRESSURE OF HIGH VOLUMES IN THE RIGHT LANES AT THE POINT BEING CONSIDERED AND WITH ROOM AVAILABLE IN OTHER LANES.

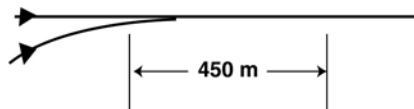
* MINIMUM % IN RIGHT LANE CANNOT BE LESS THAN % OF THROUGH TRAFFIC IN RIGHT LANE AS DETERMINED FROM TABLE 504.7C (SEE NOTE, FIG. 5047E).

** SEE FIGURE 504.2A FOR METHOD OF MEASURING LENGTH L (WEAVING LENGTH).

Figure 504.7E
Percentage of Ramp Traffic in the Outer Through Lane
(No Auxiliary Lane)
(Level of Service D Procedure)

**EXAMPLE:**

1

**A - NORMAL CALCULATION**

2 LANES ONE-WAY

"THROUGH TRAFFIC" = 2,400 VPH

"ON-RAMP" = 800 VPH

AMOUNT IN THE OUTER THROUGH LANE AT ①

THROUGH (FROM TABLE 504.7C) = $0.30 \times 2,400 = 720$ ON-RAMP (FROM CHART ABOVE) = $0.30 \times 800 = \frac{240}{960}$ **B - CHECK CALCULATIONS**

BECAUSE % IN THE OUTER THROUGH LANE AT 450 M IS BELOW DASHED LINE, RECALCULATE ASSUMING ON-RAMP TRAFFIC IS THROUGH TRAFFIC.

AMOUNT IN THE OUTER THROUGH LANE AT ①

THROUGH (FROM TABLE 504.7C) $0.40 \times 3,200 = 1,280$

SINCE CALCULATION B (1,280) IS GREATER THAN
 CALCULATION A (960) USE 1,280.

*THESE PERCENTAGES ARE NOT NECESSARILY THE DISTRIBUTIONS UNDER FREE FLOW OR LIGHT RAMP TRAFFIC, BUT UNDER PRESSURE OF HIGH VOLUMES IN THE RIGHT LANES AT THE LOCATION BEING CONSIDERED AND WITH AVAILABLE ROOM IN OTHER LANES.

NOTE: IF RAMP PERCENTAGE IN THE OUTER THROUGH LANE AT POINT UNDER CONSIDERATION IS BELOW DASHED LINE, THEN AMOUNT IN THE OUTER THROUGH LANE SHOULD BE RECALCULATED ASSUMING RAMP TRAFFIC IS THROUGH TRAFFIC. USE HIGHER VALUE. SEE EXAMPLE ABOVE.

Figure 504.8
Typical Examples of Access
Control at Interchanges

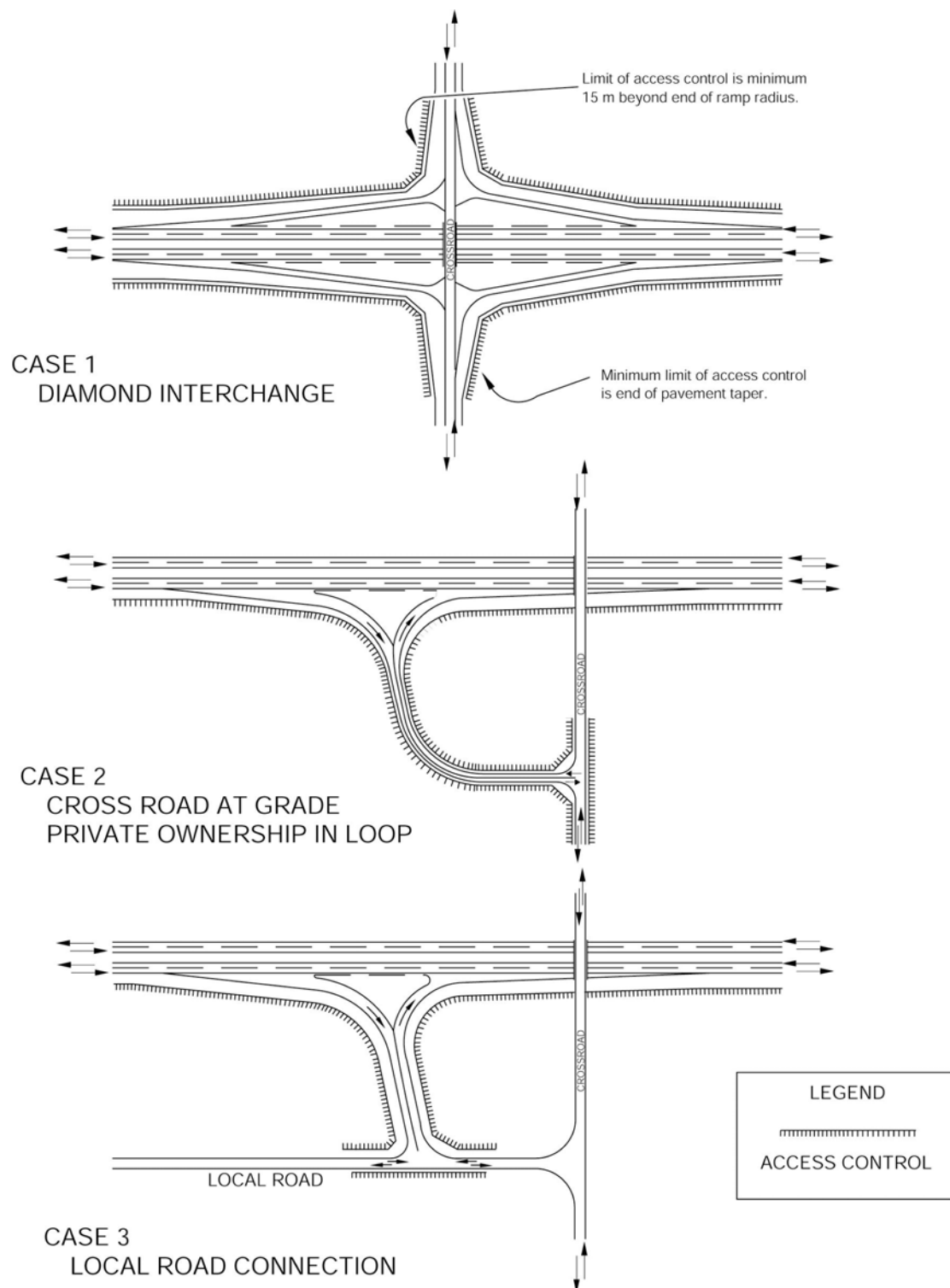


Figure 504.8 (cont.)
Typical Examples of Access
Control at Interchanges

